W-6391-03

SR 167 HOV Lanes
Renton, Washington

February 1993

Washington State Dept.
of Transportation
Attn: Mr. Todd Harrison
P.O. Box 167
Olympia, Washington 98504

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS
2722 Colby Avenue • Suite 421
Everett, Washington 98201 • 3527
206 • 252 • 5483

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

SEATTLE
EVERETT
KENNEWICK
FAIRBANKS
ANCHORAGE
SAINT LOUIS

February 22, 1993

Washington Department of Transportation
Materials Laboratory
P.O. Box 167
Olympia, Washington 98504

Attn: Mr. Todd Harrison

RE: GEOTECHNICAL REPORT, SR-167, HOV LANES, RENTON, WASHINGTON

Enclosed are 10 copies of our geotechnical report for the above project. This report supersedes our draft report of January 14, 1993 and incorporates the comments from your correspondence of January 27, 1993 and our meeting of February 11, 1993. Please call if any further clarification is needed.

We appreciate the opportunity to work with you and District 1 on these walls.

Sincerely,

SHANNON & WILSON, INC.

Paul Grant

W. Paul Grant, P.E.
Vice President

WPG/lkd

Enclosures: 10 Geotechnical Reports

W6391-03.FEB/W6391-lkd/lkd

400 NORTH 34TH STREET • SUITE 100
P.O. BOX 300303
SEATTLE, WASHINGTON 98103
206 • 632 • 8020 FAX 206 • 633 • 6777

ABSTRACT

This report presents recommendations for the design and construction of four retaining walls (Walls 6, 7, 8, and 9) on SR 167 between South 180th Street and I-405 in Renton, Washington. With the exception of Wall 6, the retaining walls will retain embankment fills which will be constructed for the High Occupancy Vehicle (HOV) lane widening of SR 167. Wall 6 will be constructed to retain an embankment cut below the west abutment of the South 180th Street overcrossing structure.

Subsurface conditions at each of the wall alignments were evaluated based upon site explorations performed specifically for this study and from the results of other explorations performed for the mainline roadway or associated facilities. These explorations indicated that the subsurface conditions in the vicinity of Wall 6 consist of medium dense to dense silty sands and very dense, till-like sediments. Cobbles were occasionally encountered in test pit excavations for this wall. The groundwater level at this wall location exists at about the level of the SR 167 roadway.

Subsurface conditions at Walls 7, 8, and 9, is generally similar in that soft, compressible surficial soils, consisting of peaty silts to clayey silts, were found overlying medium dense sands. The thickness of compressible soils at Wall 7 is on the order of 5 to 8 feet. The compressible soils beneath Walls 8 and 9 is on the order of 20 feet thick. Groundwater levels at these three retaining walls are anticipated to be located within a few feet of the existing ground surface.

In our opinion, hillside support for the cut at Wall 6 can be most readily accomplished using a soil nail wall. Specific recommendations are provided in Tables 1 and 2 for the design of this soil nail wall. Alternatively, the majority of this soil nail wall may be eliminated by site grading the adjacent hillside to a permanent slope of 2(H):1(V) and using a soil nail wall only beneath the abutment for the South 180th Street bridge. This open cut slope option, in our opinion, may be accomplished at a cost of about one-half of that for an equivalent soil nail wall. This regrading alternative would necessarily require a construction easement from the adjacent property owner.

Because of the presence of compressible and loose soils beneath Walls 7, 8, and 9, it is recommended that the embankment fill in these areas be restrained with a flexible wall system such as a Gabion wall. Other flexible wall types that would be suitable at these locations include Hilfiker (welded wire) walls, VSL/Reinforced Earth walls, geogrid/geotextile walls, criblock walls, and Keystone/Gravity Stone walls. However, it is anticipated that the Gabion wall will be the least expensive option. Additionally, use of a gabion wall system would match the

adjoining wall system that exists at the north end of Wall 9. Construction of a Gabion wall would likely require overexcavation of the soft compressible sediments beneath the wall to a depth of about 4 feet. Specific recommendations are provided for foundation overexcavation, settlement estimates, and surcharge loads.

Because overexcavation at Walls 7, 8, and 9, will not completely remove the underlying soft sediments and/or loose granular soils, these materials may experience settlement or liquefaction during a future strong earthquake. As such, the walls at locations 7, 8, and 9, may experience differential settlements on the order of several inches following a strong earthquake.

TABLE OF CONTENTS

TABLE OF CONTENTS (cont.)

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

SR 167 HOV LANES
RENTON, WASHINGTON

1.0 INTRODUCTION

This report presents the results of subsurface explorations, laboratory testing, and recommendations for the design and construction of 4 retaining walls to be constructed to accommodate High Occupancy Vehicle (HOV) lanes on State Route 167 (SR 167) in Renton between Interstate 405 and South 180th Street (Figure 1). Our geotechnical studies were conducted in accordance with Consultant Agreement Y-5266, Task Assignment #1 between Washington State Department of Transportation (WSDOT) and Shannon & Wilson, Inc. Authorization to proceed with this work was provided on December 3, 1992.

Our interpretations of subsurface conditions along the proposed wall locations were based on information contained in existing geologic maps, our field reconnaissances, subsurface explorations at or near the proposed walls, and laboratory testing results from samples retrieved from the explorations. Subsurface explorations at or near the proposed walls include 15 borings and 3 test pits which were advanced for this study and 13 borings and 3 test pits previously advanced for other WSDOT projects. Boring and test pit locations are shown on Figures 2, 7, 12, and 17. Exploration logs are presented in Appendix A; laboratory test results are presented in Appendix B. The following summarizes our findings and recommendations for each of the four retaining walls.

2.0 SITE AND PROJECT DESCRIPTION

The four proposed retaining walls (Walls 6, 7, 8, and 9) are located along a section of SR 167 that extends north-south along the east side of the Duwamish Valley, as shown on Figure 1. From its intersection with I 405 on the north, south to nearly South 180th St., SR 167 is located over the flat valley floor. The existing road grade, typically located 8 to 12 feet above the valley floor, is located on a relatively level fill embankment at an elevation of about 25 to 26 feet. At S. 180th St., the roadway cuts across a hill that extends out from the valley wall. Road cuts up to 32-feet high were made at this location to obtain the existing road grade elevations at about 27 to 28 feet.

Proposed retaining Wall 6 is located in this cut at the intersection of SR 167 and S. 180th St. As shown on Figure 2, the proposed wall is located on the west side of SR 167, approximately

83 to 95 feet west of 'L' line, between Stations 1052+12 and 1056+08. The height of soil to be retained behind this wall varies from approximately 4 to 16 feet. The slopes of the existing road cut in this area are inclined at about 2 horizontal to 1 vertical (2H:1V) and reaches a height of about 25 feet above the SR 167 road grade. At the top of the cut, the ground surface slopes back down to the west, typically at about 2H:1V. The west abutment (bent 1) of the S. 180th St. undercrossing is located on this slope, approximately 4 feet west of the proposed wall.

The remaining three walls (Walls 7, 8 and 9) are proposed to retain fill slopes along the sides of the existing roadway north of S. 180th St. The existing roadway embankments at these locations are typically inclined at a 2(H):1(V) slope to the valley floor. Lane widening to accommodate HOV traffic at these locations will require the construction of retaining walls to maintain the embankment fill within existing WSDOT right-of-way (Walls 8 and 9) or to minimize encroachment into adjacent wetlands (Wall 7). Heights for these retaining walls will vary between 2 and 12 feet.

3.0 GEOLOGY

The project is located in the Green River Trough (also referred to as the Duwamish Valley), a north-south trending valley that extends from Sumner north to Puget Sound. The trough was formed predominantly during the Vashon Stade, which began about 15,000 years ago when an ice sheet that originated in British Columbia advanced into the Puget Lowland. This trough may have existed prior to the Vashon glaciation as a shallower and narrower river valley; however, glacial scour eroded the present larger trough, as well as Puget Sound to the north. This ice sheet eroded the Green River Trough into Tertiary sedimentary rocks and deposited sediment up to hundreds of feet thick. The ice sheet retreated from the Puget Lowland about 13,500 years ago.

Rocks of the middle and upper Eocene age Tukwila Formation are exposed along the base of the hills that form the eastern margin of the Green River Trough and lie just east of the project area. These rocks are predominantly volcaniclastic sandstone, shale, conglomerate and breccia. The exposed thickness of these rocks in northwest Renton is inferred to be 2,000 to 3,000 feet. This formation probably thickens eastward and may be 10,000 feet thick 10 miles east of Renton.

Rocks of the Renton Formation, which conformably overlie the Tukwila Formation, are also exposed at the base of the hills along the eastern margin of the Green River Trough. Rocks in this formation consist predominantly of fine- to medium-grained arkosic sandstone, with lesser amounts of siltstone, shale and coal. This formation is about 2,500 feet thick near Renton.

Glacial drift sediments of the Vashon Stade overlie the Eocene bedrock in the project vicinity. The four members of the Vashon drift are (from oldest to youngest) Lawton Clay, Esperance Sand, Vashon Till and Vashon recessional deposits. Of these, Vashon Till and recessional deposits are exposed at the surface in the project vicinity.

Vashon Till, consisting predominantly of lodgement till but also including ablation till, caps the hills east and west of the Green River; forming respectively, the Covington Drift and the Des Moines Drift uplands. The lodgement till was deposited at the base of the glacial ice sheet and consists of dense to very dense, gravelly, sandy silt to silty sand with scattered cobbles and boulders. The lodgement till averages 20 feet in thickness, but exposures up to 70 feet thick have been observed. Ablation till generally overlies the lodgement till and is composed of material derived from the surface of the glacier as the ice melted. As a result, this till is thinner and less dense. Till probably occurs in the Green River Trough beneath the more recently deposited alluvium.

Recessional outwash consisting of sand and gravel was deposited in the Green River Trough during melting and retreat of the ice sheet. These deposits can include coarse outwash deltas, kame terraces and other ice-contact deposits along the valley walls, including fine sediments deposited in ephemeral ice-marginal lakes, and fluvial outwash. Also during ice sheet recession, glacial Lake Russell formed in the Duwamish Valley behind the retreating ice front. These recessional deposits may attain local thickness of 200 feet or more.

Holocene alluvium has been deposited in the Green River Trough since retreat of the ice sheet from the Puget Sound lowland, approximately 13,500 years ago. Most of this alluvium was deposited by the White River as a fan and delta. The White River originally flowed south into the Puyallup River valley. However, about 5,000 years ago, the Osceola mudflow diverted the White River to the north. The White River alluvial fan eventually extended from the Puyallup Valley north to the mouth of the Duwamish River at Seattle. This alluvium consists chiefly of sand to depths of 90 feet or more below the surface. White River overbank deposits are more abundant beginning from the surface down to a depth of about 40 feet and consist of silt, clay, fine sand and peat overlying coarse to medium sand.

An alluvial fan and delta was also formed where the Cedar River enters the Green River Trough. This deposit extends north into and beneath Lake Washington, west to the opposite valley wall and southwest beyond Longacres. At Longacres, Cedar River sediments underlie White River sediments 50 feet below the surface and continue to a depth of about 90 feet.

At present, the surface materials in the Duwamish Valley floodplain consist almost entirely of overbank deposits composed of fine sand, silt, clay and peat. Stream gradient, bedload size and channel migration along the lower Green River are significantly less than in the White River and upper Green River to the south and also significantly less than they were in the past. The lower Green River stream gradient averages about one foot per mile and the bedload consists predominantly of sand. Channel shifting caused by channel migration is slow. Flow in the Green River has been significantly reduced since 1906, when the White River was diverted back into the Puyallup River during a flood. In addition, flood peaks and sediment have been reduced by flood control structures on the Green River and channel meandering has been slowed on the lower Green river by bank armoring and flood peak reduction.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 Wall 6

4.1.1 Subsurface Conditions

Subsurface conditions in the vicinity of wall 6 were evaluated from two borings and three test pits advanced specifically for this wall and from the results of other explorations advanced for the S. 180th Street overcrossing structure and borings for the SR 167 mainline. The locations of these explorations are shown on Figure 2 and individual logs are presented in Appendix A. Generalized subsurface conditions at Wall 6 are presented in Figures 3 through 6.

As indicated on the subsurface profiles in Figures 3 through 6, the soil conditions along Wall 6 are quite variable ranging from medium dense silty sands near the top of the slope to till-like, clayey, silty sands at the toe of the slope (base of the wall). In general, most of the soils to be retained by Wall 6 include medium dense to dense silty sands with intervening layers of clayey silt to silty sand. Cobbles were encountered in the till-like stratum in Test Pit TP 6-1 and may be similarly present at other locations. The materials encountered at Wall 6 are largely thought to be ice-contact deposits.

To evaluate standup time of excavation slopes at Wall 6, Test Pit TP 6-1 was left standing for a period of about nine days before final backfilling. Essentially, no significant sloughing or failure occurred within the vertical walls of the test pit over this interval, even though water had ponded at the floor of the test pit.

The groundwater levels observed in the explorations at Wall 6 generally coincide with the base of the roadway (elevations 15 to 25 feet). However, it is noted that artesian conditions were encountered at about elevation 0 in boring H-2 which was drilled for the S. 180th street overcrossing structure. The location of the confined aquifer encountered in boring H-2 is below the planned base of the retaining wall.

4.1.2 Support Schemes

Lane widening at the S. 180th Street undercrossing (approximately Station 1055+00) will entail significant cuts into the slopes below the west abutment of the existing bridge. The west abutment is supported on spread footing foundations. The bottom of the footings are located at elevation 37 feet, which is approximately 8 feet below the bridge deck. The cut slope below the abutment is inclined at 2(H):1(V). Lane widening beneath the bridge will entail an approximate 16-foot-deep cut at the toe of the existing slope. The base of the cut will be located within about 4 feet of the abutment footing. Therefore, due to the depth of the cut and the proximity to the existing abutment, a positive and permanent excavation support system will be required to provide for the safe and continued operation of the S. 180th Street bridge.

Options for providing permanent support to the bridge abutments are limited considering the constricted work space and the close proximity of the cut to the bridge abutment. Potential construction alternatives that may be used to provide support to the cut and bridge abutment include soldier pile walls and soil nail walls. A soldier pile wall would consist of cutting holes through the bridge deck at horizontal spacings of about 5 to 8 feet to provide access for drilling and installing vertical soldier piles at the face of the west abutment. The major advantage of this scheme is that the soldier piles may be incorporated as underpinning piles for the abutment. This would reduce the risk of footing settlement during construction. A major disadvantage of this scheme, however, is that it would require the closure of S. 180th Street during the installation of the underpinning piles. Another disadvantage is that large (approximately 3 feet in diameter) access holes would need to be cut in the bridge deck. This would essentially entail deck removal and reconstruction near the abutment and additional delays for the bridge closure. Because construction of a soldier pile wall may require lengthy closure of the S. 180th Street bridge and costly renovation of a portion of the bridge deck, this is not a preferred support system for Wall 6.

In our opinion, a soil nail wall may meet the objectives of providing uninterrupted access on S. 180th Street and still provide an adequate level of support for the abutment. Since a soil nail wall does not require vertical soldier piles, it may be installed below the existing bridge without interrupting traffic. A disadvantage of this system, however, is the fact that movement of the wall is required to develop the tensile resistance of the soil nails. This wall movement will necessarily result in movement (settlement) of the west abutment of the bridge. Since the greatest amount of movement is expected to occur at the face of the wall, it is necessary to provide adequate clearance between the wall and the abutment to reduce potential objectional settlements.

In early February, a meeting was held involving WSDOT Headquarters Materials Laboratory, District 1 and Shannon & Wilson to address concerns about the potential performance of the S. 180th Street bridge west abutment and applicable support schemes for Wall 6. As a result of this meeting it was decided that a soil nail wall would be the most practical method of providing support to the cut below the west abutment. However, to reduce the risk of potential objectional movements of the abutment footing, it was agreed that the wall should be placed no closer than 4 feet from the edge of the footing.

While there will still be some risk of footing movement with the soil nail wall, this movement potential may be addressed with various precautionary measures such as detailed excavation and construction sequencing procedures and prestressing soil nails beneath the abutment. Also monitoring points should be established on the bridge abutment to document actual settlement during construction with respect to anticipated levels of movement. Finally, remedial underpinning measures may be undertaken during construction if movements of the abutment are found to be excessive. Such underpinning measures could include drilling high capacity piers to support the ends of the abutment with a single pile installed at the centerline of the roadway, in the 4-foot-wide zone between the wall and the bridge abutment footing.

Considering the use of the precautionary design and construction measures mentioned above and the potential remedial options available for support, it is our opinion that a soil nail wall would provide the most cost-effective support for the abutment. Details of the wall design and recommended construction sequencing are discussed below.

4.1.3 Soil Nail Wall Design

Design recommendations for the soil nail wall were developed using the computer program SNAIL which was developed by the California Department of Transportation

(CALTRANS). This program uses a bi-linear wedge analysis to evaluate the stability of the soil nail reinforced cut slope. The program directly computes factors of safety against slope failure for the specified soil nail spacing, length, and load transfer capacity. Our analyses were conducted to develop a design that would provide a minimum factor of safety for static loading of 1.5 and a factor of safety for dynamic loading of at least 1.1. The dynamic factor of safety was computed based upon a pseudo-static stability analysis using the SNAIL program and a horizontal seismic coefficient of 0.15g. This seismic coefficient is consistent with values that are commonly used by the Corps of Engineers in the seismic evaluation of earth dams in the Pacific Northwest.

The analyses considered that the wall sections would retain an approximate 2H:1V backslope. For those portions of the wall that would be influenced by the bridge abutment, the analyses also included an estimated bridge abutment surcharge load of 5 ksf. The analyses also modeled the water table at the base of the wall.

Design recommendations for the soil nail wall are summarized in Tables 1 and 2. Table 1 provides design recommendations for the portion of the wall beyond the abutment while Table 2 provides design recommendations for the portion of the wall below the abutment. Specific details are provided in these tables including the soil nail grid pattern, the location of the top row of nails, nail inclination, bar size, length, and design load. Integral to this design is the assumed length and allowable load transfer capacity (i.e. soil-nail bond stress) of the soil nails. The assumed allowable load transfer capacities of the soil nails were selected considering the materials encountered in the subsurface explorations in the proposed vicinity of the wall. The contractor must be required to install nails to the minimum lengths indicated on Tables 1 and 2, and demonstrate through verification tests that the nails have a soil-nail bond stress (load transfer capacity) of at least twice the allowable values indicated on these tables. This should be accomplished by performing two verification tests. These tests should be conducted on soil nails within 20 feet of the bridge abutment that are installed at the same elevation as the first row of soil nails at the abutment.

Verification (and proof) test nails will require a no-load zone, a bonded or test length, and may require additional steel area to prevent bar failure. The no-load zone should be a minimum of 5 feet for test loads less than 10 kips and 10 feet for test loads greater than or equal to 10 kips. The test length should be determined by dividing the nail design load (kips) by the allowable load transfer value (kips/foot). All verification tests should be performed so that the installed nail has an ultimate load transfer value of at least twice (factor of safety of 2) the allowable values indicated on Tables 1 and 2. If the soil nails installed by the Contractor are not able to meet this performance objective, the Contractor would need to install longer nails that are capable of supporting the design loads with a factor of safety of at least 2.0. During the installation of the production nails, at least 5 percent of these nails should be proof-tested to 150 percent of the design loads. Test loads are defined as the Capacity Test Load for verification and proof tests as described in the general special provisions for soil nail walls developed by WSDOT. All soil nails must be epoxy-coated in accordance with the AASHTO requirements M284/M284 M-87.

Because of the proximity of the soil nail wall to the west abutment of the bridge, it is recommended that all 4 rows of the soil nails between stations 1054+55 and 1055+64 be preloaded to 25 percent of the design load. This preloading would tend to reduce potential wall movements and settlement of the adjacent footing. However, the shotcrete would have to achieve the required strength prior to preloading the soil nails. It is recommended that the nails be preloaded by hydraulic jacking, similar to the methods used to test the verification and proof test nails.

We recommend that prefabricated drainage materials be installed at the face of the wall as the excavation proceeds. It is recommended that 1-foot-wide segments of Miradrain 6000, Amerdrain 520, or the equivalent, be installed between each column of soil nails. The drainage material should be continuous for the entire height of the wall. Each drain should be connected to weep holes (min. 2-inch-diameter) daylighting through the shotcrete facing at the base of the wall. Commercial grate connectors are recommended to link the drainage material to the weep holes. Additionally, it is recommended that a subdrain be installed at the base of the wall and connected to the drainage material to further facilitate the removal of groundwater. The subdrain should be connected to the roadway storm drain system.

Wall 6 will require both a temporary shotcrete facing and a permanent concrete fascia panel. In our opinion, various alternatives are available for the design of the wall facing. First, the wall facing may be designed based upon a uniform pressure of 500 psf, with individual support points provided at the soil nail locations. Alternatively, the wall facing may be designed based upon recommendations recently developed by CALTRANS. In completing the design of the shotcrete facing, it is recommended that the wall be capable of resisting the punching force of the soil nails as indicated in Tables 1 and 2. This punching resistance was an integral assumption in the SNAIL analysis of the walls.

It is estimated that settlement and horizontal movement at the top of the wall would be approximately 1/2-inch. Settlement of the west abutment footing is anticipated to be on the order of 1/4 to 3/8 inches provided the recommendations presented in Section 5.2 are followed. These settlement estimates are based upon empirical experience with soil nail walls in the Seattle area.

4.1.4 Instrumentation

To confirm the satisfactory performance of the soil nail wall and to provide a system of detecting adverse performance of the bridge abutment, we recommended that four survey points be established across west abutment of the South 180th Street bridge. These survey points should be established at the end points of the abutment footings and at the third points along the exposed abutment footings beneath the bridge deck. These survey points should be monitored for both horizontal and vertical movement on a weekly basis during the construction of the soil nail wall. If horizontal or vertical movements of the abutment are observed to be in excess of 1/2-inch, construction of the soil nail walls should be halted to determine the source of this movement and to establish the type and extent of any remedial construction.

4.1.5 Slope Grading Alternative

As an alternative to constructing a soil nail wall for the full length section of Wall 6, it may be economically advantageous to construct a soil nail wall directly below the bridge abutment (Station 1054+55 to 1055+64) and to construct open cuts on the slope both north and south of the overcrossing structure. As shown on Figures 3 and 5, 2(H):1(V) slopes could be constructed in lieu of the soil nail wall over the majority of the proposed length of Wall 6. It is estimated that the constructed cost of a soil nail wall with a fractured fin fascia panel would be on the order of $300 per linear foot of wall whereas the excavation of the adjacent slope may be accomplished for approximately $150 per linear foot of the proposed wall. With some minor exceptions, this suggested regrading would be accomplished entirely within WSDOT right-of-way. Only the portion of the slope grading between Station 1052+25 and 1054+55 would extend beyond the WSDOT right-of-way. This would necessarily require right-of-way construction easements to accomplish this grading on the adjacent property. Additionally, there would appear to be a utility pole that may require re-location as a result of this site grading.

4.2 Wall 7

4.2.1 Subsurface Conditions

Subsurface conditions in the vicinity of Wall 7 were evaluated from eight borings drilled specifically for this wall and from the results of other explorations that were performed for the SR 167 mainline roadway. The locations of these explorations are shown on Figure 7 and individual boring logs are presented in Appendix A. Generalized subsurface conditions at Wall 7 are presented in Figures 8 through 11.

As depicted in Figure 11, which is a longitudinal profile along Wall 7, the southern quarter of the wall will be constructed over native soils consisting of very loose to loose silty fine sands which extend to about elevation 8 feet. These loose soils are underlain by medium dense silty fine to medium sands. The northern portion of the wall will likely be founded upon the embankment fill from the SR 167 roadway. This embankment fill extends to an elevation of approximately 15 feet. It appears that this embankment fill directly overlies an 8-foot thick stratum of soft peaty silt and clayey peat. This stratum of silt and peat similarly overlies the medium dense, silty sand stratum at about elevation 8 feet.

The native soils observed in the borings advanced along Wall 7 are interpreted as having been deposited as overbank flood deposits from the Green River. The peaty soils and underlying clay observed underlying the northern portion of the wall were formed in a marsh area on the floodplain that was periodically inundated with flood waters. The underlying medium dense sands are volcaniclastic in origin and may have been deposited as White River alluvial fan sediments.

The static groundwater level along Wall 7 exists at relatively shallow depths. Within the southern 250-foot section of the wall, the groundwater table is anticipated to be within several feet of the ground surface. North of this point, the groundwater level is essentially at the ground surface where the existing SR 167 roadway crosses a freshwater marsh.

4.2.2 Design Recommendations

Since the majority of the length of Wall 7 will be constructed over compressible sediments or loose soils, it is recommended that a flexible wall system be used to provide restraint to the embankment fill. This wall system would be required to provide a grade separation ranging from 2 to 6 feet. The most economical means of providing this support, in our opinion, would be through the construction of a Gabion wall. However, other wall types may be similarly considered as alternatives for this location. Other wall types could include a Hilfiker (welded wire) wall system, a VSL/Reinforced Earth wall, criblock wall, a Gravity Stone/Keystone wall, and geogrid/geotextile walls. With the exception of Gravity Stone/Keystone walls and geogrid/-geotextile walls, all of the above wall types have been pre-approved by WSDOT. In our opinion, a conventional concrete cantilever wall is not recommended at this location because of the underlying compressible soils. Use of a concrete cantilever wall would necessarily require installation of piling to provide support for the wall.

The following provides specific recommendations for construction of a flexible wall system (i.e., gabion wall) at the location of Wall 7:

A. Soils to a depth of 4 feet below the base of wall should be removed and replaced with Gravel Backfill for Foundations, Class 4, (GBF) WSDOT Standard Specifications 9-03.12(1)A between Sta. 1076+00 to Sta. 1081+70. An overexcavation depth of 3 feet is recommended from sta. 1075+60 to Sta. 1076+00, with no overexcavation from Sta. 1081+70 to Sta. 1083+50. Overexcavations should extend from the back face of the wall to the toe of existing embankment. The minimum width of overexcavation is 8 feet (extending from the back face of the wall). The purpose of the overexcavation is to remove soft organic soils and provide adequate bearing capacity for wall foundations, lower the magnitude of immediate and long-term settlements, and increase stability or lateral sliding resistance of the slope.

   Excavation of soft organic soils could be accomplished in the wet with a backhoe and the excavation should be immediately backfilled with GBF. This procedure, if accomplished properly, would avoid the need for shoring. In our opinion, the backfill may be end dumped without compacting for portions of the fill located below the water table. Above the water table, the fill should be compacted to 95 percent of maximum density (WSDOT Standard Specification 2-03.3(14)C, Method C).

   Stability analyses of the wall with the recommended foundation overexcavation indicated that the proposed design would have a minimum factor of safety of 1.5.

B. Settlement would be primarily a function of the thickness of the underlying soft organic layers directly beneath the wall. At Wall 7 this thickness appears to vary from 3 to approximately 7 feet. Settlement of Wall 7 is estimated to range from 2 to 4 inches and could occur over a period of 3 to 5 months.

   To accelerate settlement, the embankment fill could be surcharged with an amount of additional fill equal to half the maximum fill height. Surcharge is not recommended from Sta. 1075+60 to Sta. 1076+50 and Sta. 1081+00 to Sta. 1083+50 because fill heights are relatively low. The surcharge fill should occupy the width of the HOV lane widening. We estimate settlements would take place in 15 to 30 days and should be monitored with a series of settlements plates. Measurements should be taken weekly. Plates should be embedded near the base of the wall and spaced at 100-foot intervals. This settlement estimate should only be used for preliminary planning purposes. The removal of the surcharge load, however, should be based on the field data from the settlement plate readings. This may require adjustments within the construction schedule to accommodate potentially longer preload requirements.

C. We recommend an allowable bearing pressure of 3,000 pounds per square foot (psf) for the wall design. This pressure may be increased by 1/3 for transient loading. A friction coefficient of 0.5, which includes a factor of safety of 1.5, should be used to calculate sliding resistance. Lateral fluid densities of 30 pcf and 42 pcf are recommended for permanent and short-term surcharge loading conditions, respectively. These pressures reflect a level ground surface behind the wall. A minimum traffic surcharge of 2 feet is recommended. In other words, the lateral pressure with surcharge for a 6-foot wall should be based on a total height of 8 feet. Alteratively, the retained backfill and the fill soil at the base of the wall may be characterized by a unit weight of 125 pcf and an internal friction angle of 34 degrees with an appropriate pressure increase for traffic surcharge.

D. Since native loose sand may be present below the Gabion wall, there is a potential that these soils may liquify during a strong earthquake. Liquefaction of these sediments may result in several inches of settlement of the wall.

4.3 Wall 8

4.3.1 Subsurface Conditions

Subsurface conditions in the vicinity of Wall 8 were evaluated from three borings drilled specifically for this study and from the results of other explorations advanced for the SR 167 mainline roadway and associated facilities. The locations of these explorations are shown on Figure 12. Individual logs of these explorations are presented in Appendix A. Generalized subsurface conditions along Wall 8 are provided in Figures 13 through 16.

Subsurface conditions along Wall 8, as depicted in Figure 16, indicate that the wall alignment may be underlain by a surficial stratum of fill soils that may have been placed in conjunction with the construction of the adjacent frontage road and associated utilities. These fill soils overlie a layer of loose silty sand to soft clayey, peaty silt which extends to an elevation of approximately 5 feet. Below these compressible soils, medium dense sands were encountered in the site explorations which typically terminated between elevations 0 and -10 feet.

The fill soils encountered in the explorations may not necessarily underlie the proposed location of Wall 8, considering that the explorations may have been advanced through fill of the adjacent frontage road. Therefore, the surficial soils at the location of Wall 8 may consist of soft compressible sediments that extend to about elevation 5 feet. These compressible sediments may be 15 to 20 feet thick at the location of Wall 8.

The groundwater level was typically observed in the site explorations within about 5 feet of the existing ground surface. However, groundwater was encountered at a depth of 14 feet at boring location B 8-2.

4.3.2 Design Recommendations

Since Wall 8 will be constructed over compressible sediments or loose soils, it is recommended that a flexible wall system be used to provide restraint to the embankment fill. This wall system would be required to provide a grade separation ranging from 3 to 12 feet. The most economical means of providing this support, in our opinion, would be through the construction of a Gabion wall. However, other wall types may be similarly considered as alternatives for this location. Other wall types could include a Hilfiker (welded wire) wall system, a VSL/Reinforced Earth wall, criblock wall, a Gravity Stone/Keystone wall, and geogrid/geotextile walls. With the exception of Gravity Stone/Keystone walls and geogrid/geotextile walls, all of the above wall types have been pre-approved by WSDOT. In our opinion, a conventional concrete cantilever wall is not recommended at this location because of the underlying compressible soils. Use of a concrete cantilever wall would necessarily require installation of piling to provide support for the wall. Alternatively, open fill slopes inclined at 2(H):1(V) may be used for embankment construction provided that this fill remains within the WSDOT right-of-way. This alternative would particularly apply north of Station 980+00.

The following provides specific recommendations for construction of a flexible well system (i.e., gabion wall) at the location of Wall 8:

A. To provide a suitable foundation for the wall and reduce potential settlements, the soils to a depth of 4 feet below the base of wall should be removed and replaced with GBF. This overexcavation would apply over the entire length of the wall. The overexcavation should extend 15 feet behind the face of the wall.

Excavation of soft organic soils could be accomplished in the wet with a backhoe and the excavation should be immediately backfilled with GBF. This procedure, if accomplished properly, would avoid the need for shoring. In our opinion, the backfill may be end dumped without compacting for portions of the fill located below the water table. Above the water table, the fill should be compacted to 95 percent of maximum density (WSDOT Standard Specification 2-03.3(14)C, Method C).

Stability analyses of the wall with the recommended foundation overexcavation indicated that the proposed design would have a minimum factor of safety of 1.5.

B. The extent of compressible soft organic soils are more prevalent at Wall 8 compared to Wall 7. Consequently, larger settlements are anticipated and the time to the end of primary consolidation would be longer. The potential also exists for settlement of the adjacent East Valley Road.

Settlements at Wall 8 are estimated to range from 4 to 8 inches. To accelerate these settlements, we recommend placing a 4-foot surcharge over the full width of the HOV lane widening. Settlements from this surcharge are estimated to occur in 30 to 60 days and should be monitored with a series of settlement plates embedded at the base of the wall. The plates should be spaced at 100-foot intervals and readings obtained on a weekly basis. This settlement estimate should only be used for preliminary planning purposes. The removal of the surcharge load, however, should be based on the field data from the settlement plate readings. This may require adjustments within the construction schedule to accommodate potentially longer preload requirements.

C. We recommend an allowable bearing pressure of 3,000 pounds per square foot (psf) for the wall design. This pressure may be increased by 1/3 for transient loading. A friction coefficient of 0.5, which includes a factor of safety of 1.5, should be used to calculate sliding resistance. Lateral fluid densities of 30 pcf and 42 pcf are recommended for permanent and short-term surcharge loading conditions, respectively. These pressures reflect a level ground surface behind the wall. The equivalent fluid weight should be increased to 45 pcf to reflect a 2(H):1(V) permanent slope behind the wall. A minimum traffic surcharge of 2 feet is recommended. In other words, the lateral pressure with surcharge for a 6-foot wall should be based on a total height of 8 feet. Alteratively, the retained backfill and the fill soil at the base of the wall may be characterized by a unit weight of 125 pcf and an internal friction angle of 34 degrees with an appropriate pressure increase for traffic surcharge.

D. Since native loose sand may be present below the wall, there is a potential that these soils may liquify during a strong earthquake. Liquefaction of these sediments may result in several inches of settlement of the wall.

4.4 Wall 9

4.4.1 Subsurface Conditions

Subsurface conditions in the vicinity of Wall 9 were evaluated based upon the results of two explorations drilled specifically for this wall and from the results of other explorations advanced in the area. Locations of these explorations are shown on Figure 17. Individual logs from these explorations are presented in Appendix A. Generalized subsurface conditions encountered in the explorations at Wall 9 are presented in Figures 18 through 20.

Subsurface conditions along wall 9, which are depicted in Figure 20, indicate that the wall alignment is underlain by approximately 5 to 8 feet of soft clayey silt which overlies a stratum of very loose, silty, fine sand which extends to about elevation 0. Below this stratum, medium dense sands were encountered in exploration HB 9-2. These medium dense sands may exist at a somewhat lower elevation further north toward the end of Wall 9.

The groundwater level along Wall 9 is anticipated to be located within 2 feet of the existing ground surface.

4.4.2 Design Recommendations

Since Wall 9 will be constructed over compressible sediments or loose soils, it is recommended that a flexible wall system be used to provide restraint to the embankment fill. This wall system would be required to provide a grade separation ranging from 7 to 10 feet. The most economical means of providing this support, in our opinion, would be through the construction of a Gabion wall similar to the existing wall at the north end of Wall 9 which was constructed in 1988. However, other wall types may be similarly considered as alternatives for this location. Other wall types could include a Hilfiker (welded wire) wall system, a VSL/Reinforced Earth wall, criblock wall, a Gravity Stone/Keystone wall and geogrid/textile walls. With the exception of Gravity Stone/Keystone walls and geogrid/geotextile walls, all of the above wall types have been pre-approved by WSDOT. In our opinion, a conventional concrete cantilever wall is not recommended at this location because of the underlying compressible soils. Use of a concrete cantilever wall would necessarily require installation of piling to provide support for the wall. Alternatively, open fill slopes, inclined at 2(H):1(V), may be used for embankment construction provided this fill remains within the WSDOT right-of-way. This alternative would particularly apply south of Station AR2 80+00.

The following provides specific recommendations for construction of a flexible wall system (i.e., gabion wall) at the location of Wall 9:

A. To provide a suitable foundation for the wall and reduce potential settlements, the soils to a depth of 4 feet below the base of wall should be removed and replaced with GBF. This overexcavation would apply to the entire length of the wall. The overexcavation should extend out from the wall face 4 feet or to the ROW whichever is less. To the east, the overexcavation should extend 4 feet beyond the wall.

   Excavation of soft organic soils could be accomplished in the wet with a backhoe and the excavation should be immediately backfilled with GBF. This procedure, if accomplished properly, would avoid the need for shoring. In our opinion, the backfill may be end dumped without compacting for portions of the fill located below the water table. Above the water table, the fill should be compacted to 95 percent of maximum density (WSDOT Standard Specification 2-03.3(14)C, Method C).

   The adjacent building should be monitored for movement during the excavation phase. If the building settles or moves laterally shoring should be provided. We recommend a review of the construction reports and monitoring records of the existing Gabion wall so that we can re-evaluate Wall 9 recommendations, particularly settlement estimates and consolidation times.

   Stability analyses of the wall with the recommended foundation overexcavation indicated that the proposed design would have a minimum factor of safety of 1.5.

B. Wall/embankment settlements are estimated to range from 3 to 5 inches where soft organic soils exist, and less between Sta. AR2 80+00 and Sta. AR2 82+00. A 4-foot surcharge is recommended between Sta. AR2 79+50 to Sta. AR2 83+00 to accelerate the foundation settlements. The surcharge should occupy the width of the HOV lane widening. Settlements from the surcharge load are estimated to occur within 30 to 60 days. We recommend the adjacent building be surveyed for movement and visually inspected prior to construction. Existing cracks or other features should be documented. This settlement estimate should only be used for preliminary planning purposes. The removal of the surcharge load, however, should be based on the field data from the settlement plate readings. This may require adjustments within the construction schedule to accommodate potentially longer preload requirements.

C. We recommend an allowable bearing pressure of 3,000 pounds per square foot (psf) for the wall design. This pressure may be increased by 1/3 for transient loading. A friction coefficient of 0.5, which includes a factor of safety of 1.5, should be used to calculate sliding resistance. Lateral fluid densities of 30 pcf and 42 pcf are recommended for permanent and short-term surcharge loading conditions, respectively. These pressures reflect a level ground surface behind the wall. The equivalent fluid pressures should be increased to 45 pcf to reflect a 2(H):1(V) permanent slope behind the wall. A minimum traffic surcharge of 2 feet is recommended. In other words, the lateral pressure with surcharge for a 6-foot wall should be based on a total height of 8 feet. Alteratively, the retained backfill and the fill soil at the base of the wall may be characterized by a unit weight of 125 pcf and an internal friction angle of 34 degrees with an appropriate pressure increase for traffic surcharge.

D. Since native loose sand may be present below the wall, there is a potential that these soils may liquify during a strong earthquake. Liquefaction of these sediments may result in several inches of settlement of the wall.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 Excavations

Construction of the Walls 6, 7, 8, and 9, will entail excavating native soils ranging from very soft peats and clays to very dense, till-like glacial sediments which may include cobbles and boulders. In our opinion, conventional excavating equipment may be generally used to accomplish the excavations. However, excavations for Walls 7, 8, and 9, will most readily be accomplished using a track hoe to overexcavate the foundation soils in the wet. Fill placement within this excavated zone could similarly be accomplished using a track hoe or by means of pushing fill into the excavation with a small bulldozer. An 18- to 24-inch initial lift thickness may be required over the soft underlying sediments to provide a stable base for operation of compaction equipment. It is recommended that the Contractor be responsible for the safety of all temporary excavation slopes that may be required to construct these retaining walls.

5.2 Soil Nailing

In our opinion, the plans and specifications for the construction of the soil nail wall should incorporate the general special provisions that are currently being developed by WSDOT for the construction of soil nail walls.

The soil nail wall should be constructed using top down construction techniques. Using this system, an excavation is typically made for a row of soil nails, drilling is accomplished for the nail, the nail is installed and grouted, and shotcrete is placed over the face of the exposed excavation. This sequence is repeated for successive rows of nails until the wall is completed.

Observations at test pit TP 6-1 indicate soils should stand unsupported for a limited time. However, to minimize raveling of the excavation face, if it occurs, the Contractor may need to provide a shallow stabilizing berm at the excavation face. Soil nails would then be installed through this berm. This berm must lie below the previous shotcreted lift and be constructed at a safe slope. After installation of the nails, the berm should be removed without damaging the nails.

Because of the close proximity of Wall 6 to the west abutment of the S. 180th Street bridge, special construction sequencing will be required to reduce the risk of potential loss of ground beneath the abutment. Recommended construction procedures for this section of wall below the bridge abutment are outlined on Figure 21. It is recommended to include these procedures in the contract specifications.

It is our understanding that a storm drain line will be constructed within about 3 feet horizontally and 4 feet vertically from the base of Wall 6. Considering that this drain will be installed below the base of Wall 6, it is recommended that the drain construction be conducted limiting open trenches to no more than 20 feet in length. Alternatively, trench boxes should be used adjacent to the wall during the drain installation to provide lateral support to the hillside.

Scattered boulders and cobbles were encountered in our the subsurface explorations at the general location of Wall 6. Therefore, it is recommended that the contract specifications contain an advisory statement indicating that cobbles and boulders will likely be encountered in the excavations and in the drilling of the soil nail holes. The presence of these materials may require installation of additional soil nails or altering construction procedures if obstructions are encountered. Additionally, the Contractor should be prepared to remove any cobbles or boulders that protrude into the soil face of the excavation and backfill the void with shotcrete. In addition, groundwater may be encountered in the nails installed near the base of the wall. Different drilling techniques and additional verification tests should be required for soil nails located below groundwater.

5.3 Construction Monitoring

We recommend that an experienced geotechnical engineer be retained to provide construction monitoring services for the project. These services will be required to evaluate the suitability of the bearing stratum exposed at the base of the wall footing overexcavations, interpreting settlement plate readings for surcharge removal, and the adequacy of soil nail installations. Considering that this drain will be installed below the base of Wall 6, it is recommended that the drain construction be conducted limiting open trenches to no more than 20-foot lengths. Alternatively, trench boxes should be used adjacent to the wall to provide lateral support for the hillside. Observation of the installation of soil nails is particularly critical to the assure that they satisfy the design loading requirements.

6.0 LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based upon site conditions as the presently exist and further assume that the borings are representative of subsurface conditions throughout the site, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the field explorations or inferred from geologic maps and site reconnaissances.

If, during construction, subsurface conditions different from those encountered in the field explorations are observed or appear to be present beneath excavations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between the submission of this report and start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, it is recommended that this report be reviewed to determine the applicability of these conclusions and recommendations, considering the changed conditions and the elapsed time.

Shannon & Wilson has prepared the attachment in Appendix C, "Important Information About Your Geotechnical Engineering Report", to assist you and others in understanding the use and limitations of our reports.

We recommend that we be retained to review the geotechnical portions of the plans and specifications, to determine if they are consistent with our recommendations. In addition, we should be retained to monitor geotechnical aspects of constructions, particularly soil nail installation and testing.

This report is prepared for the exclusive use of the Washington State Department of Transportation for the design of proposed retaining wall 6, 7, 8 and 9. It should be made available to prospective contractors and/or the contractor for information on factual data only, and not as a warranty of subsurface conditions described in this report.

Please note that the scope of our services did not include any environmental assessment or evaluation regarding the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around the SR-167 alignment.

Unanticipated soil conditions are commonly encountered and cannot fully be determined by merely taking soil samples from test borings. Such unexpected conditions frequently require that additional expenditures be made to attain properly constructed projects. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

SHANNON & WILSON, INC.

Jeffrey R. Laird
Geologist

William J. Perkins
Engineering Geologist

THOMAS M. GURTOWSKI
STATE OF WASHINGTON
16945
REGISTERED
PROFESSIONAL ENGINEER
2/22/93
EXPIRES 9/29/ 93

W. PAUL GRANT
STATE OF WASHINGTON
20099
REGISTERED
PROFESSIONAL ENGINEER
2-22-93
EXPIRES 11/10/ 94

Thomas M. Gurtowski, P.E.
Associate

W. Paul Grant, P.E.
Vice President

JRL:WJP:TMG:WPG/wjp

2-20-93/W6391-03.RPT/W6391-lkd/dgw

1.57

TABLE 1

SOIL-NAIL DESIGN
WALL 6 - BEYOND ABUTMENT

	15.5' ≥ Wall Height > 13'		13' ≥ Wall Height > 10'		10' ≥ Wall Height > 6'		6' ≥ Wall Height	
Approx. Stationing	1052+95 to 1054+55 and 1055+64 to 1055+85		1052+39 to 1052+95 and 1055+85 to 1055+93		1052+19 to 1052+39 and 1055+93 to 1056+04		1052+12 to 1052+19 and 1056+04 to 1056+08	
Soil Conditions								
Material type	Ice-Marginal Gravel, Sand and Silt		Ice-Marginal Gravel, Sand and Silt		Ice-Marginal Gravel, Sand and Silt		Ice-Marginal Gravel, Sand and Silt	
Unit Weight (pcf)	125		125		125		125	
Friction Angle (°)	38		38		38		38	
Cohesion (pcf)	0		0		0		0	
Soil Nails								
Pattern (ft)	6(H) x 4(V)		6(H) x 4(V)		6(H) x 4(V)		6(H)	
Nail Inclination (°)	15		15		15		15	
Nail Diameter (inches)	6		6		6		6	
Location of First Row (below top of retained soil - ft)	3		3		4		3	
Soil-Nail Bond Stress (psi)	2600 psf 18[1]	1200 psf 8.3[2]	18[1]	8.3[2]	18[1]	8.3[2]	18[1]	8.3[2]
Load Transfer (kips/ft)	4.1	1.9	4.1	1.9	4.1	1.9	4.1	1.9
Bar Size (Grade 60-ASTM A-706)								
Row 1 (top)	No. 6	No. 7	No. 5	No. 6	No. 5	No. 6	No. 6	No. 6
Row 2	No. 7	No. 7	No. 6	No. 6	No. 7	No. 7	- / -	- / -
Row 3	No. 9	No. 9	No. 8	No. 8	- / -	- / -	- / -	- / -
Minimum Length (ft)[3]								
Row 1 (top)	17	26	13	19	11	17	8	9
Row 2	17	26	12	18	10	16	- / -	- / -
Row 3	17	26	12	18	- / -	- / -	- / -	- / -
Design Load (Kips-Static/Dynamic [0.15g])[3]								
Row 1 (top)	7.7 / 0.0	18.5 / 20.6	3.0 / 3.7	11.5 / 14.5	5.8 / 4.0	13.5 / 17.1	8.6 / 15.7	6.8 / 9.0
Row 2	18.0 / 15.1	23.2 / 27.1	12.8 / 15.9	15.4 / 19.2	18.9 / 23.1	17.7 / 22.4	- / -	- / -
Row 3	31.6 / 35.5	26.7 / 32.0	26.4 / 32.7	20.6 / 25.5	- / -	- / -	- / -	- / -
Wall Punching								
Capacity (kips)	40	40	40	40	40	40	40	40
Stability Factor of Safety[3]								
Static	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Dynamic	1.2	1.2	1.2	1.2	1.2	1.2	1.1	1.2

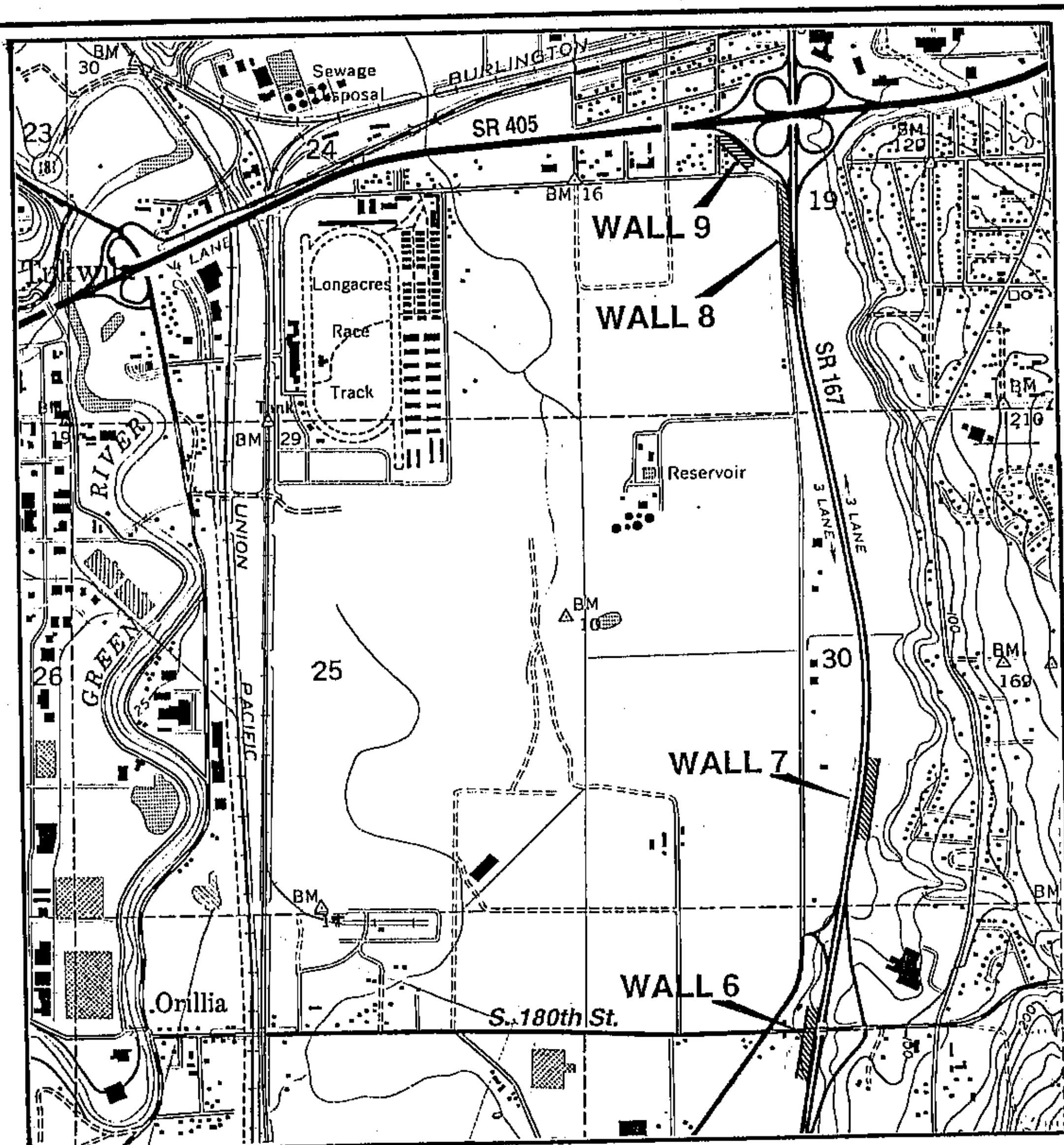
NOTES:
1. 18 psi soil-nail bond stress corresponds to a 6-inch diameter pressure-grouted anchor
2. 8.3 psi soil-nail bond stress corresponds to a 6-inch diameter, low-pressure grouted anchor
3. Reported anchor lengths, loads, and Factors of Safety for 6" diameter, pressure-grouted (18 psi soil-nail bond stress) anchors are essentially the same for 12" diameter, low pressure grouted (8.3 psi soil-nail bond stress) anchors.

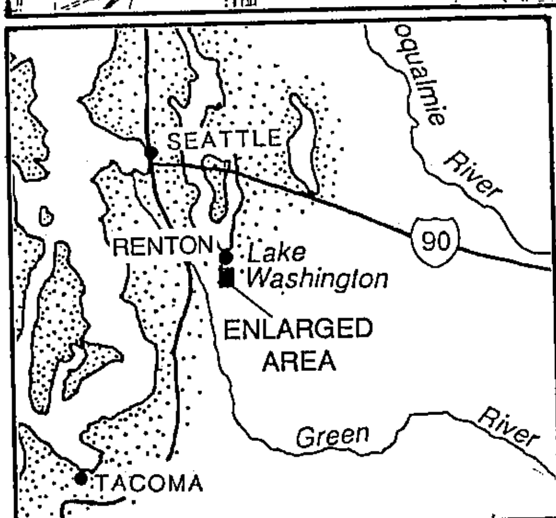
TABLE 2

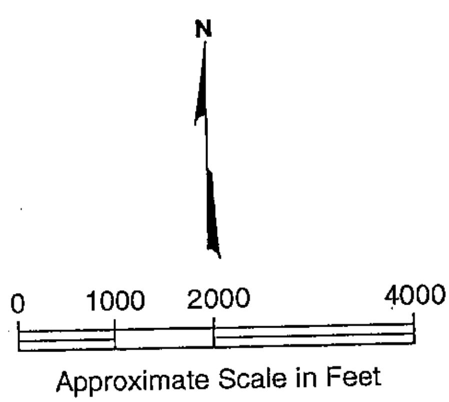
SOIL-NAIL DESIGN
WALL 6 - BELOW ABUTMENT OF S. 180th ST.

Approx. Stationing	1054+55 to 1055+64	
Soil Conditions		
Material type	Ice-Marginal Gravel, Sand and Silt	
Unit Weight (pcf)	125	
Friction Angle (°)	38	
Cohesion (pcf)	0	
Top of Retained Soil		
Elev. (ft)	Approx. 40.5 to 39 feet	
Soil Nails		
Pattern (ft)	4(H) x 4(V)	
Nail Inclination (°)	15	
Nail Diameter (inches) *	6	
Soil-Nail Bond Stress (psi)	**18** [1]	**8.3** [2]
Load Transfer (kips/ft)	4.1	1.9
Nail Elevation at Wall Face (ft)		
Row 1 (top)	36	36
Row 2	32	32
Row 3	28	28
Row 4	25	25
Bar Size	1" diameter Dywidag (all rows)	
Minimum Length (ft) [3]		
Row 1 (top)	27	51 [4]
Row 2	22	35 [4]
Row 3	20	35 [4]
Row 4	20	35 [4]
Design Load (Kips-Static/Dynamic [0.15g]) [3]		
Row 1 (top)	54.5 / 53.2	55.6 / 58.8
Row 2	42.8 / 42.3	35.5 / 38.0
Row 3	36.3 / 40.2	36.0 / 39.8
Row 4	39.1 / 43.2	37.3 / 41.2
Wall Punching		
Capacity (kips)	60	70
Stability Factor of Safety [3]		
Static	1.5	1.5
Dynamic	1.3	1.3

NOTES:
1. 18 psi soil-nail bond stress corresponds to a 6-inch diameter pressure-grouted anchor
2. 8.3 psi soil-nail bond stress corresponds to a 6-inch diameter, low-pressure grouted anchor
3. Reported anchor lengths, loads, and Factors of Safety for 6" diameter, pressure-grouted (18 psi soil-nail bond stress) anchors are essentially the same for 12" diameter, low pressure grouted (8.3 psi soil-nail bond stress) anchors. 65 error 1055+64
4. For the wall segment between stations 1054+55 and ~~1054+75~~, the soil nails extend beyond WSDOT rigth-of-way and would require a construction easement from the adjacent property owner.



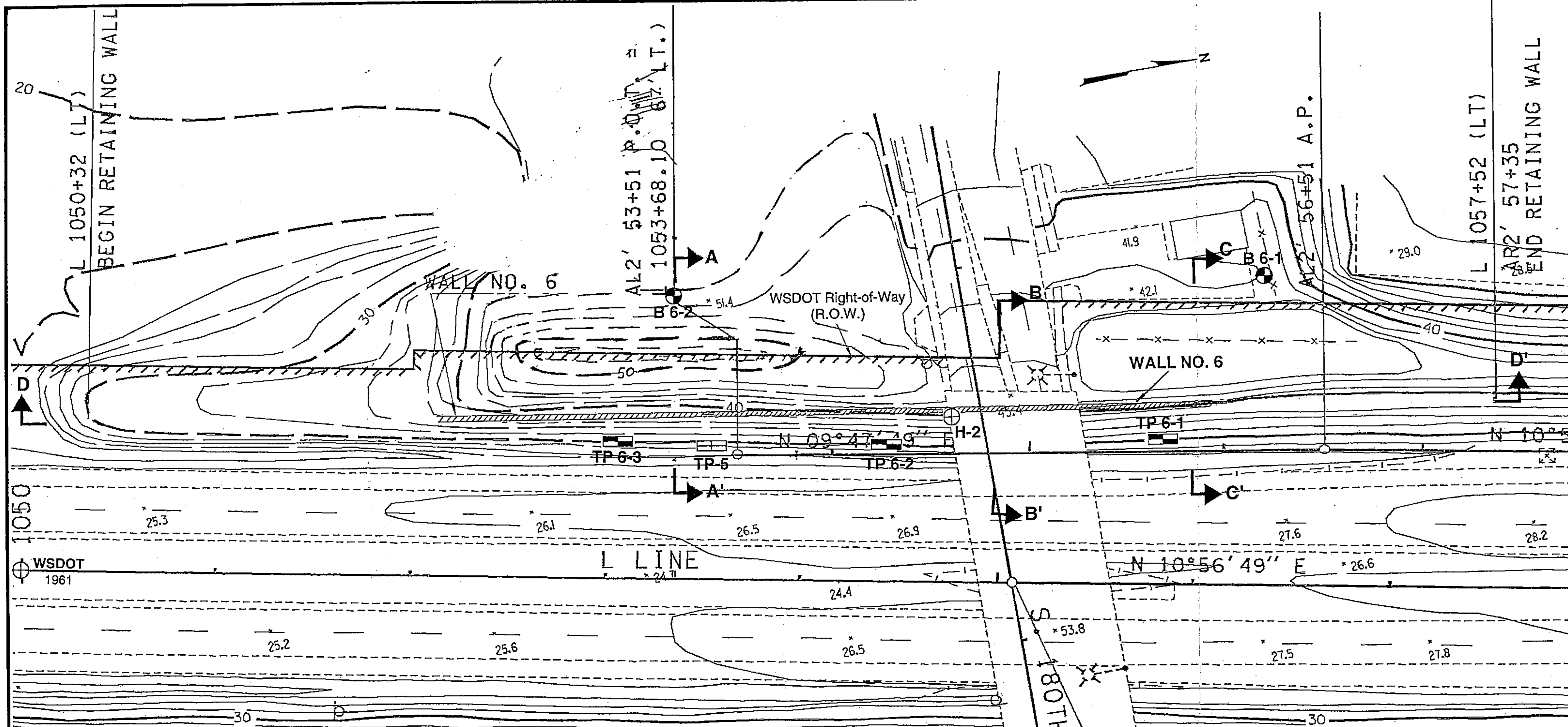




NOTE

Vicinity map adapted from USGS 7.5' Topographic Map of Renton quadrangle, dated 1973.

SR167 H.O.V. Lanes Renton, Washington	
PROJECT VICINITY MAP AND WALL LOCATIONS	
January 1993	W-6391-03
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 1**



LEGEND

B 6-2 — S & W Boring Designation and Approximate Location Completed for this Study

TP 6-3 — S & W Test Pit Designation and Approximate Location Completed for this Study

WSDOT 1961 H-2 — Boring Designation and Approximate Location Completed by Others for Previous Studies

TP-5 — Test Pit Designation and Approximate Location Completed by Others for Previous Studies

Proposed Retaining Wall

A — Generalized Subsurface Profile

0 50 100

Scale in Feet

NOTES

1. Base map provided by WSDOT. Revised topography provided by Shannon & Wilson.
2. Contour intervals are two feet.

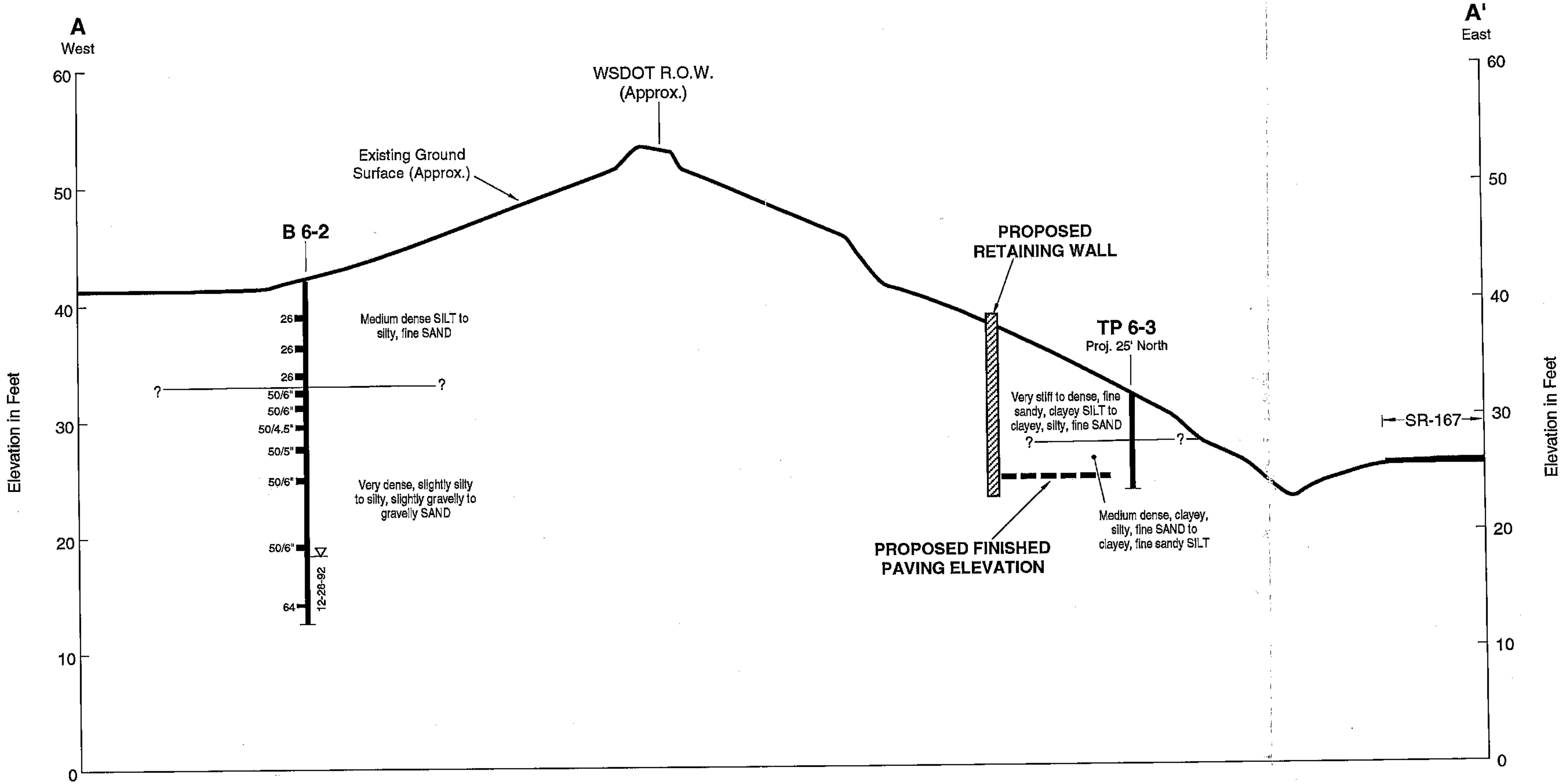
SR167 H.O.V. Lanes
Renton, Washington

SITE AND EXPLORATION PLAN
WALL NO. 6

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 2



0 10 20

Scale in Feet
Horizontal = Vertical

LEGEND

B 6-2 ← S&W Boring Location and Designation

TP 6-3 ← S&W Test Pit Location and Designation

Proj. 25' North ← Offset Distance

12-28-92 ← Groundwater Level and Date Recorded

7 ← Sample Taken During Boring, Standard Penetration Resistance in Blows/Foot or Blows/Inches Driven

?—? Approximate Geologic Contact

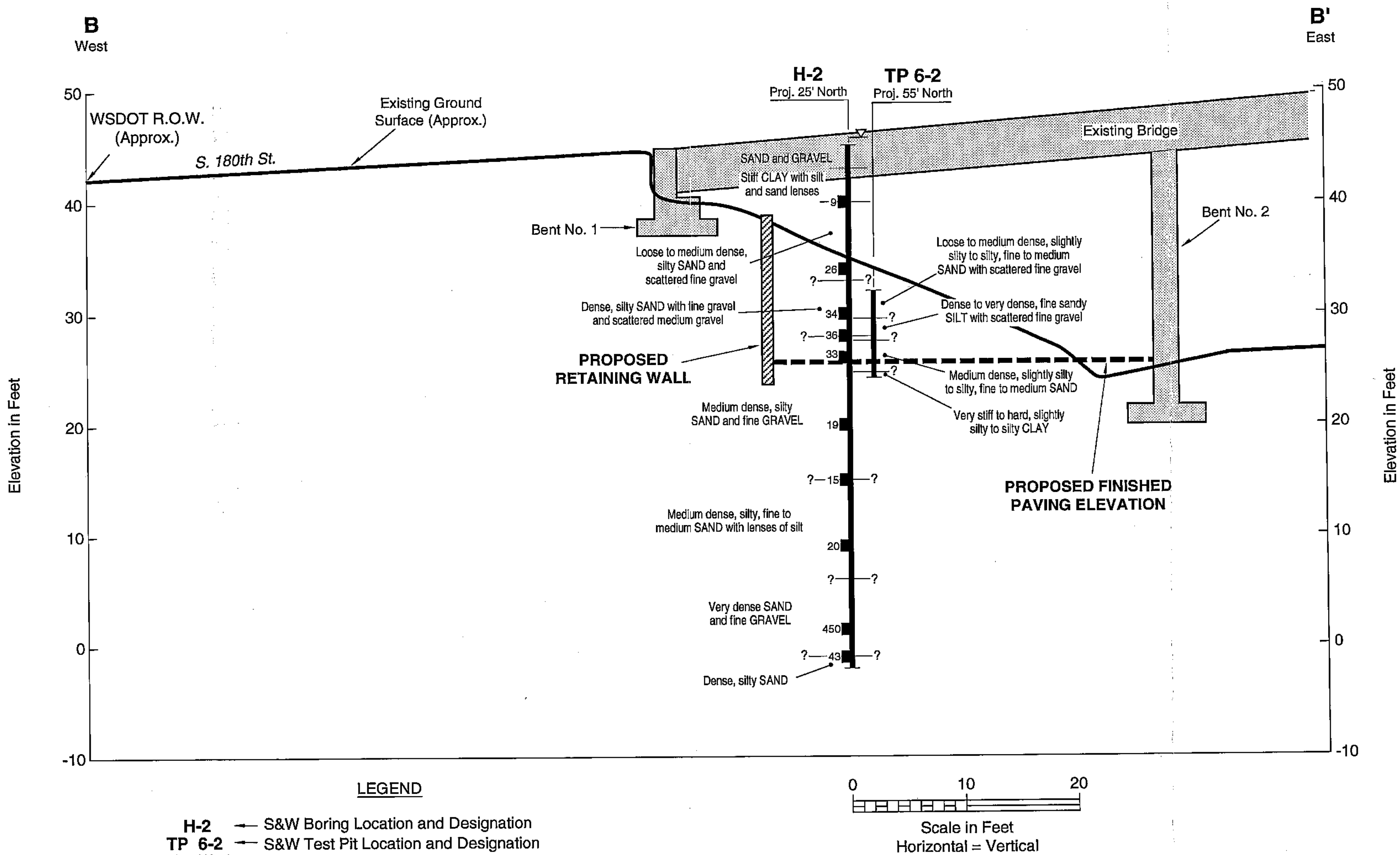
← Bottom of Boring

NOTES

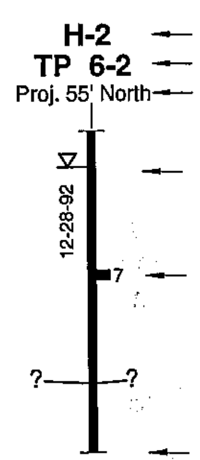
1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.

2. Refer to individual boring and test pit logs in Appendix A for details of each boring.

SR 167 H.O.V. Lanes Renton, Washington	
SUBSURFACE PROFILE A-A' **WALL NO. 6** **AR2 1053+35**	
January 1993	W-6391-03
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 3**



LEGEND



- S&W Boring Location and Designation
- S&W Test Pit Location and Designation
- Offset Distance
- Groundwater Level and Date Recorded
- Sample Taken During Boring, Standard Penetration Resistance in Blows/Foot or Blows/Inches Driven
- Approximate Geologic Contact
- Bottom of Boring

NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.
2. Refer to individual boring and test pit logs in Appendix A for details.

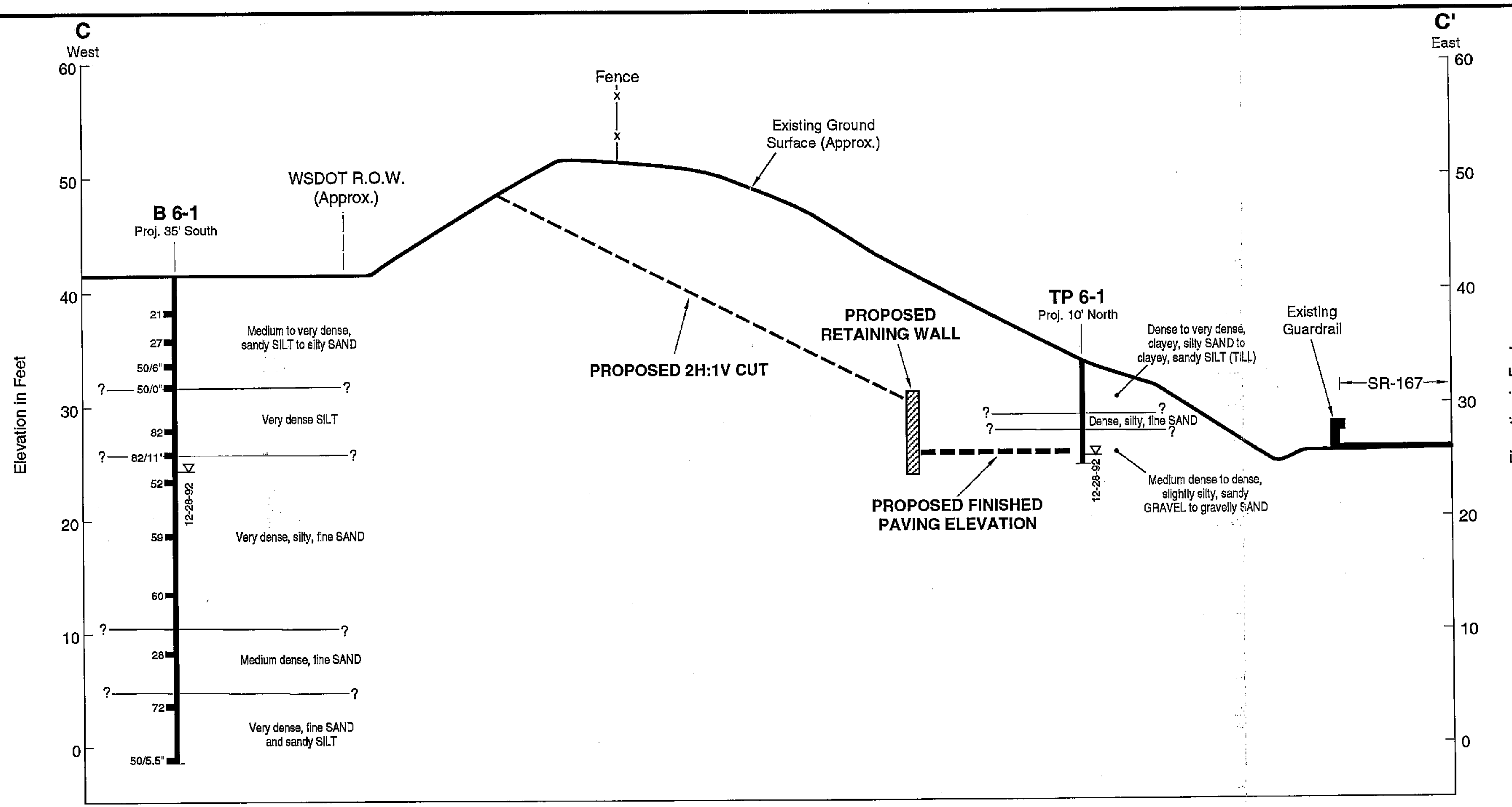
SR 167 H.O.V. Lanes
Renton, Washington

SUBSURFACE PROFILE B-B'
WALL NO. 6
AR2 1055+00

January 1993 W-6391-03

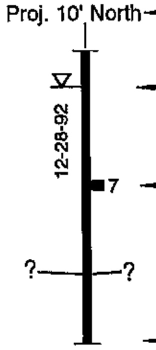
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 4



LEGEND

B 6-1 — S&W Boring Location and Designation
TP 6-1 — S&W Test Pit Location and Designation
Proj. 10' North — Offset Distance

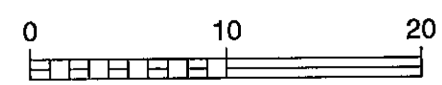


Groundwater Level and Date Recorded

Sample Taken During Boring, Standard Penetration Resistance in Blows/Foot or Blows/Inches Driven

Approximate Geologic Contact

Bottom of Boring



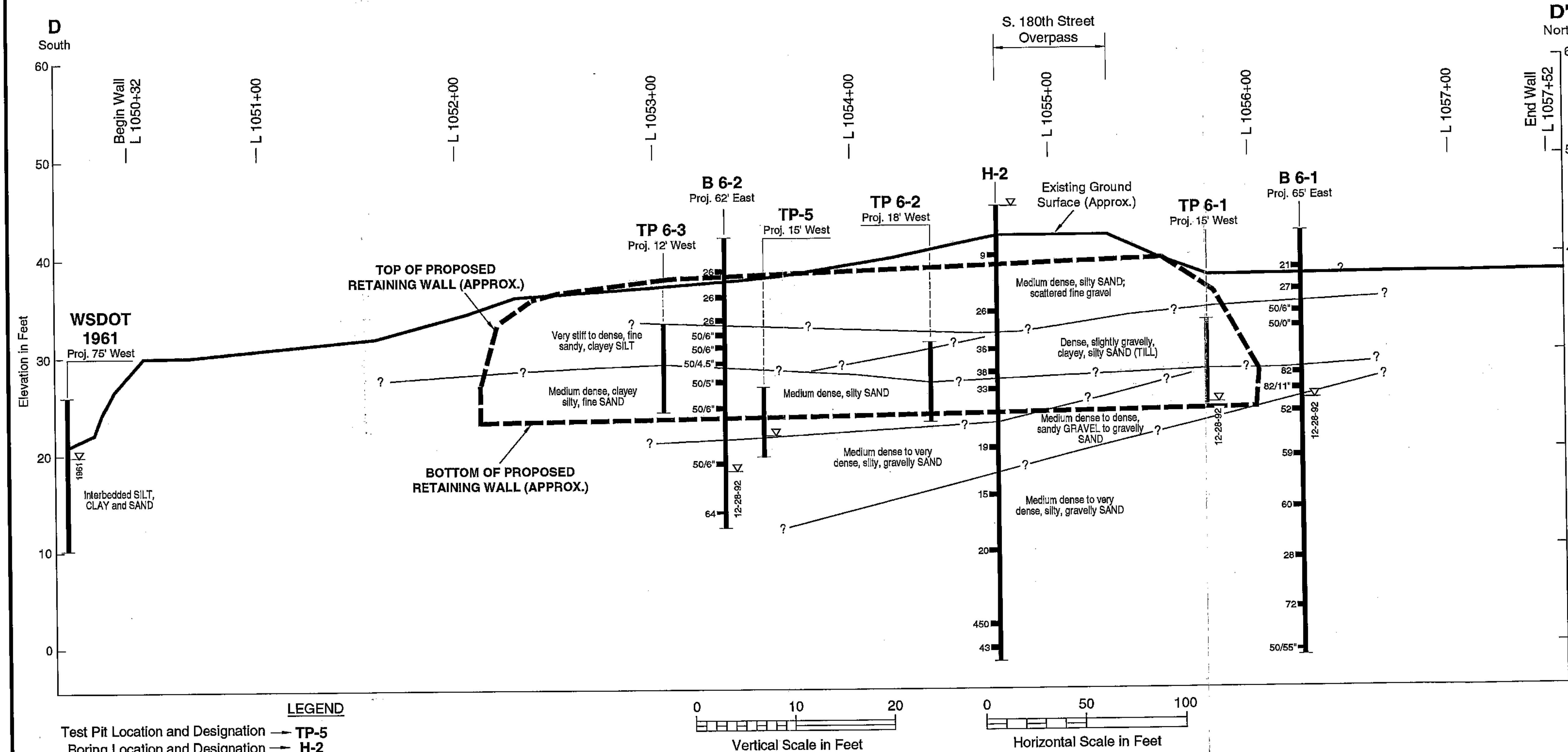
Scale in Feet
Horizontal = Vertical

NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.

2. Refer to individual boring and test pit logs in Appendix A for details.

SR 167 H.O.V. Lanes Renton, Washington	
SUBSURFACE PROFILE C-C' WALL NO. 6 AR2 1056+00 January 1993 W-6391-03	
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	FIG. 5



LEGEND

Test Pit Location and Designation — TP-5
Boring Location and Designation — H-2
(Completed by Others for Previous Studies)
B 6-1 — S&W Boring Location and Designation
TP 6-1 — S&W Test Pit Location and Designation
Proj. 18' West — Offset Distance
12-31-92 Groundwater Level and Date Recorded
7 or 50/4" Sample Taken During Boring, Standard Penetration Resistance in Blows/Foot or Blows/Inches Driven
?—? Approximate Geologic Contact
Bottom of Boring

NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.

2. Profile as drawn has a 5x vertical exaggeration.

3. Refer to individual boring and test pit logs in Appendix A for details of each boring.

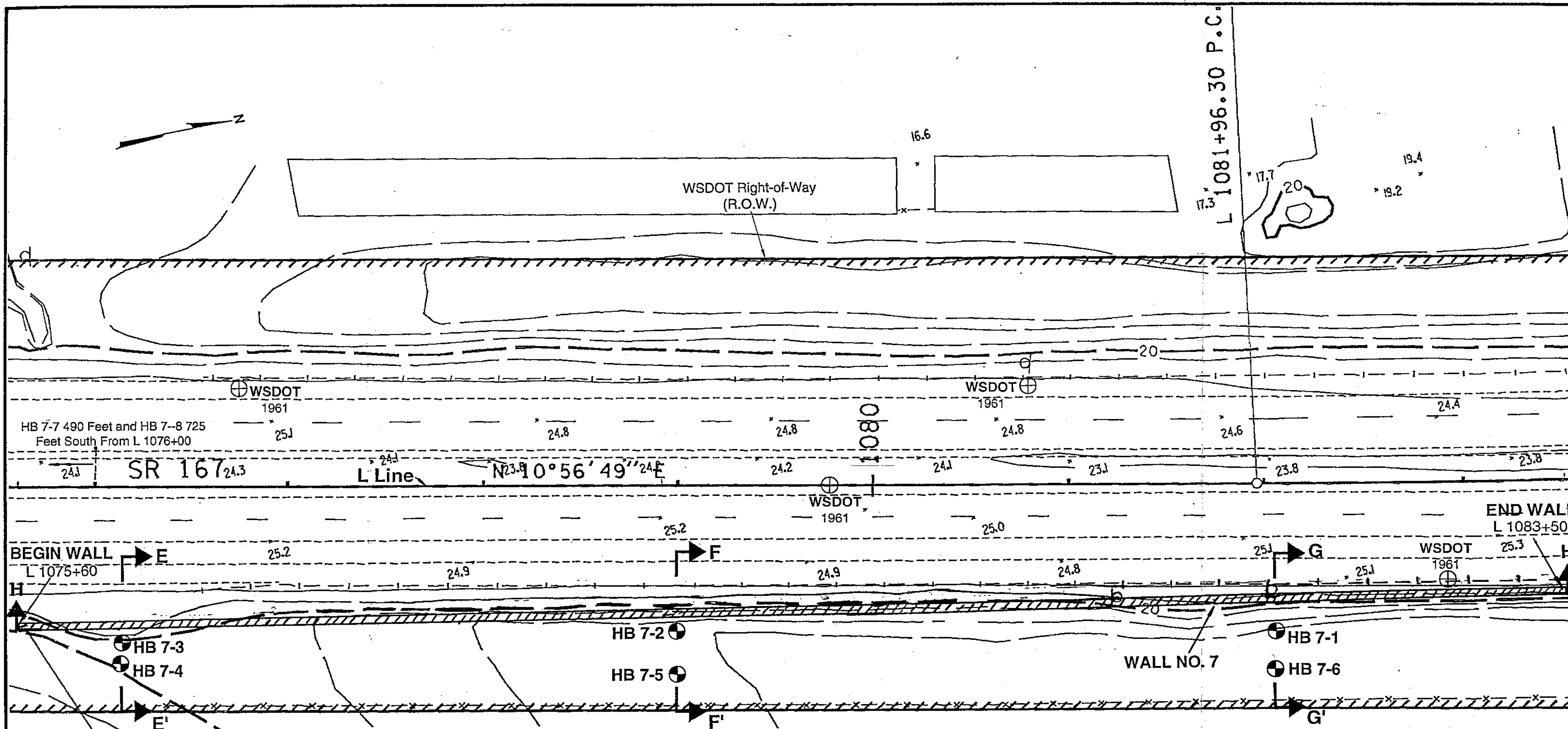
SR 167 H.O.V. Lanes
Renton, Washington

SUBSURFACE PROFILE D-D'
WALL NO. 6
L 1050+00 TO 1057+60

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 6



LEGEND

HB 7-3	S & W Boring Designation and Approximate Location Completed for this Study
WSDOT 1961	Boring Designation and Approximate Location Completed by Others for Previous Studies
	Proposed Retaining Wall
E	Generalized Subsurface Profile

0 50 100

Scale in Feet

NOTES

1. Base map provided by WSDOT.
2. Contour intervals are two feet.

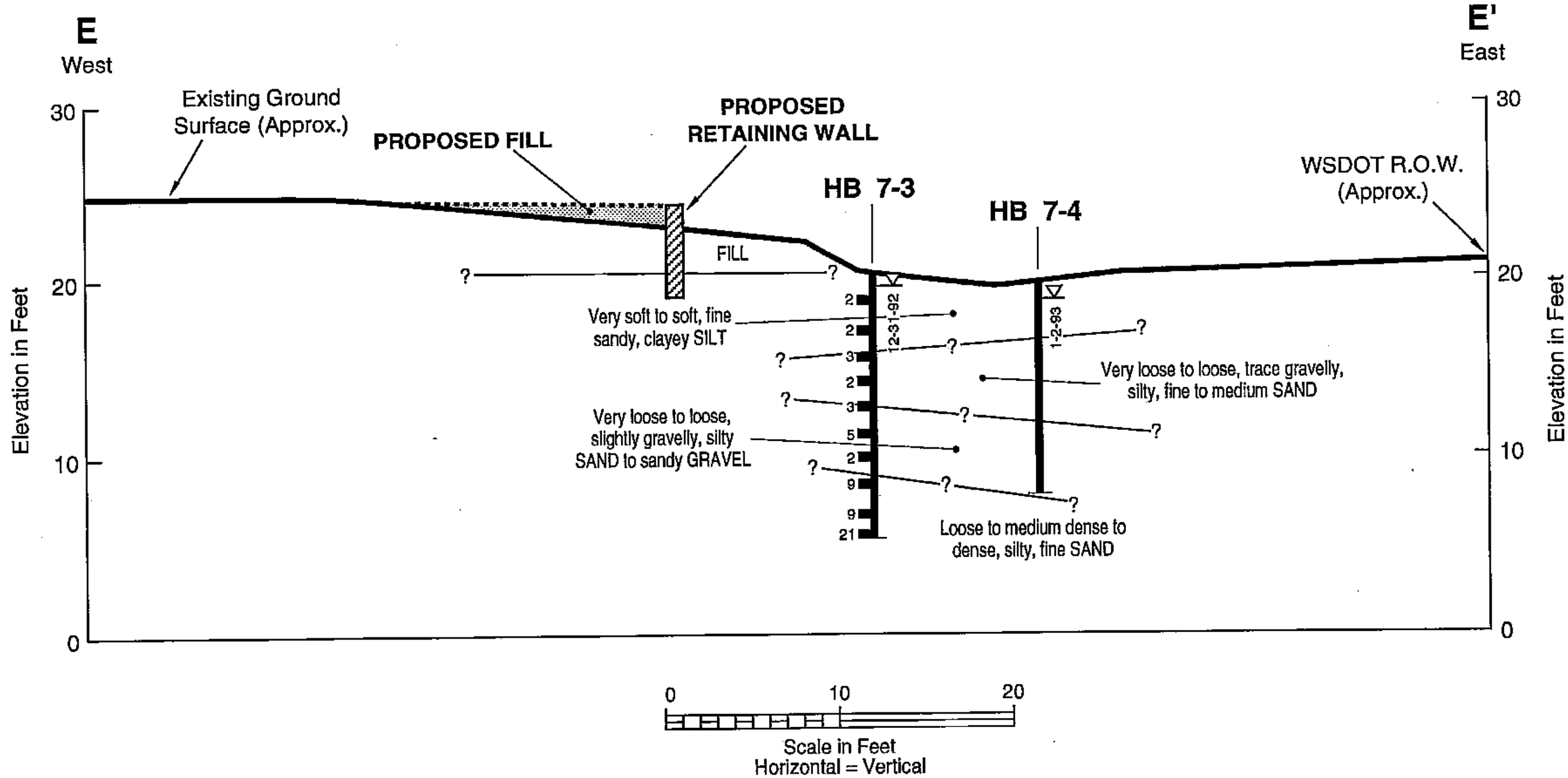
SR167 H.O.V. Lanes
Renton, Washington

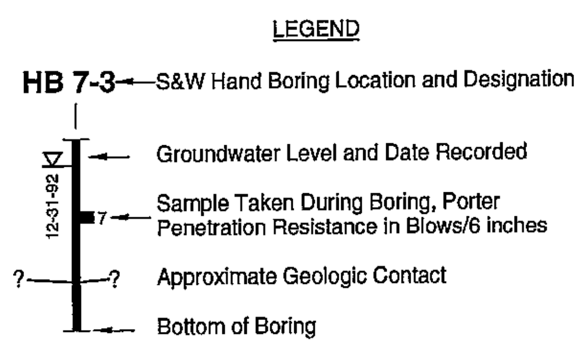
SITE AND EXPLORATION PLAN
WALL NO. 7

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 7





NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.
2. Refer to individual boring logs in Appendix A for details of each boring.

SR 167 H.O.V. Lanes
Renton, Washington

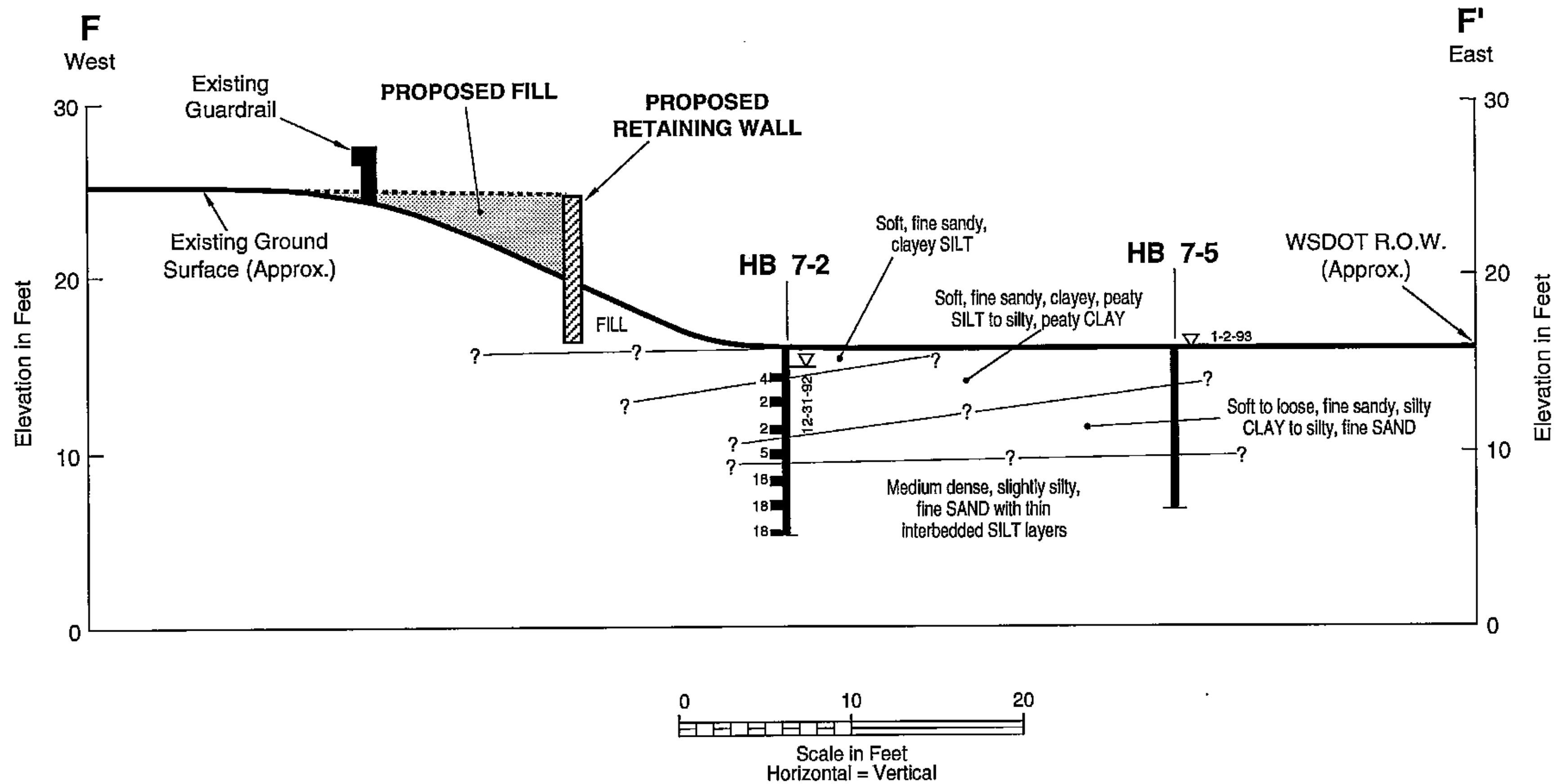
SUBSURFACE PROFILE E-E'
WALL NO. 7
L 1076+15

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 8

FIG. 8



LEGEND

HB 7-2 — S&W Hand Boring Location and Designation

Groundwater Level and Date Recorded

Sample Taken During Boring, Porter Penetration Resistance in Blows/6 Inches

Approximate Geologic Contact

Bottom of Boring

NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.
2. Refer to individual boring logs in Appendix A for details of each boring.

SR 167 H.O.V. Lanes
Renton, Washington

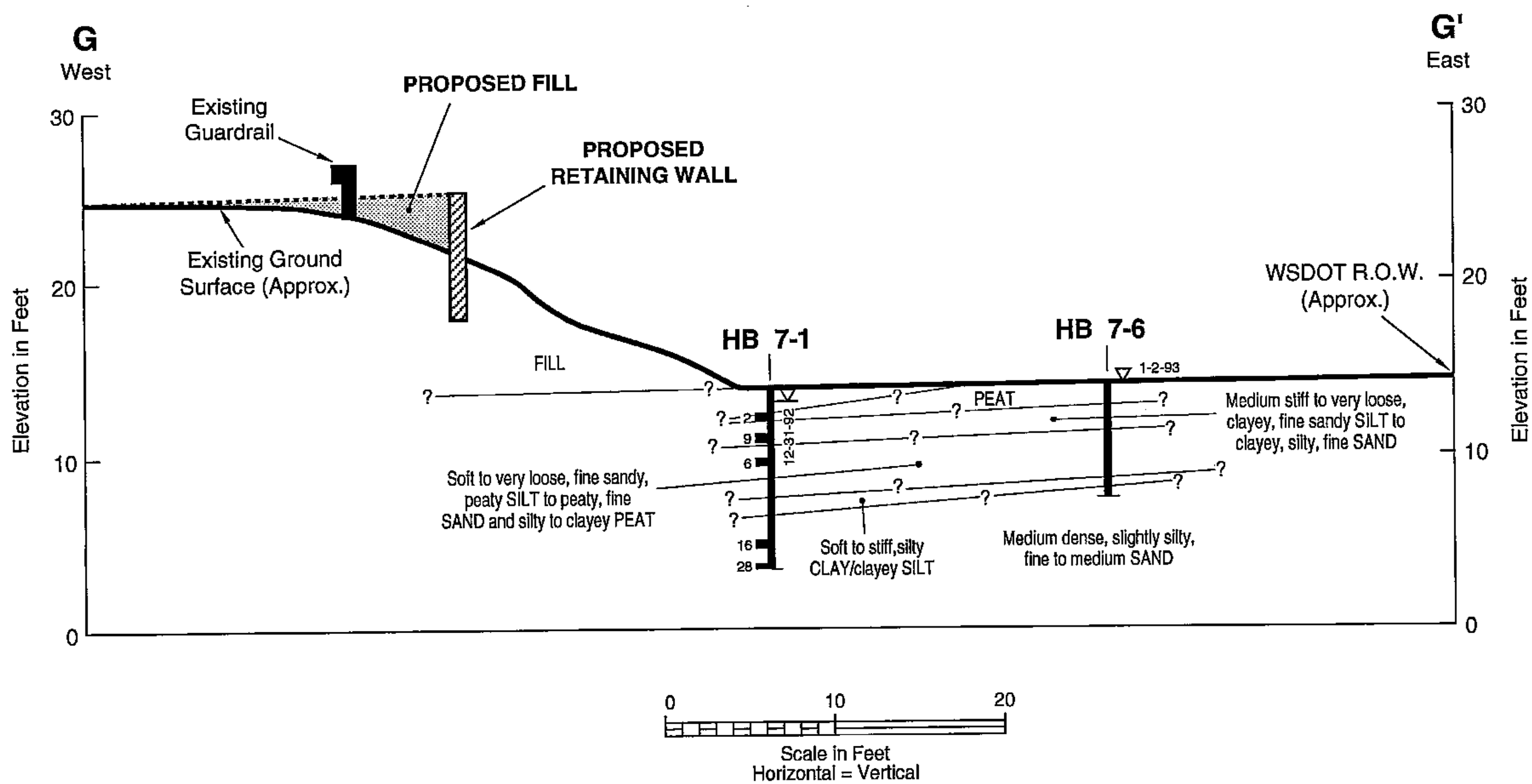
SUBSURFACE PROFILE F-F'
WALL NO. 7
L 1079+00

January 1993 W-6391-03

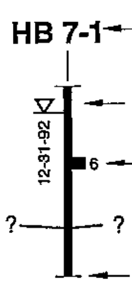
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 9

FIG. 9



LEGEND



HB 7-1 — S&W Hand Boring Location and Designation

Groundwater Level and Date Recorded

Sample Taken During Boring, Porter Penetration Resistance in Blows/6 Inches

Approximate Geologic Contact

Bottom of Boring

NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.

2. Refer to individual boring logs in Appendix A for details of each boring.

SR 167 H.O.V. Lanes
Renton, Washington

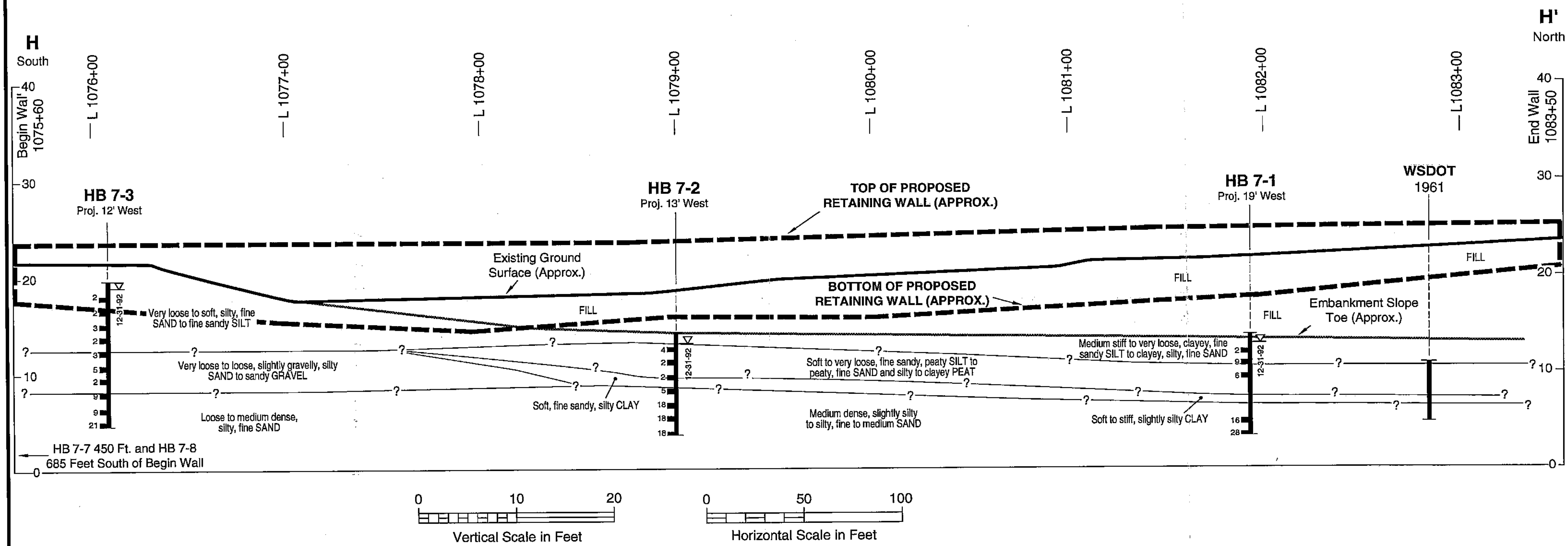
SUBSURFACE PROFILE G-G'
WALL NO. 7
L 1082+05

January 1993 W-6391-03

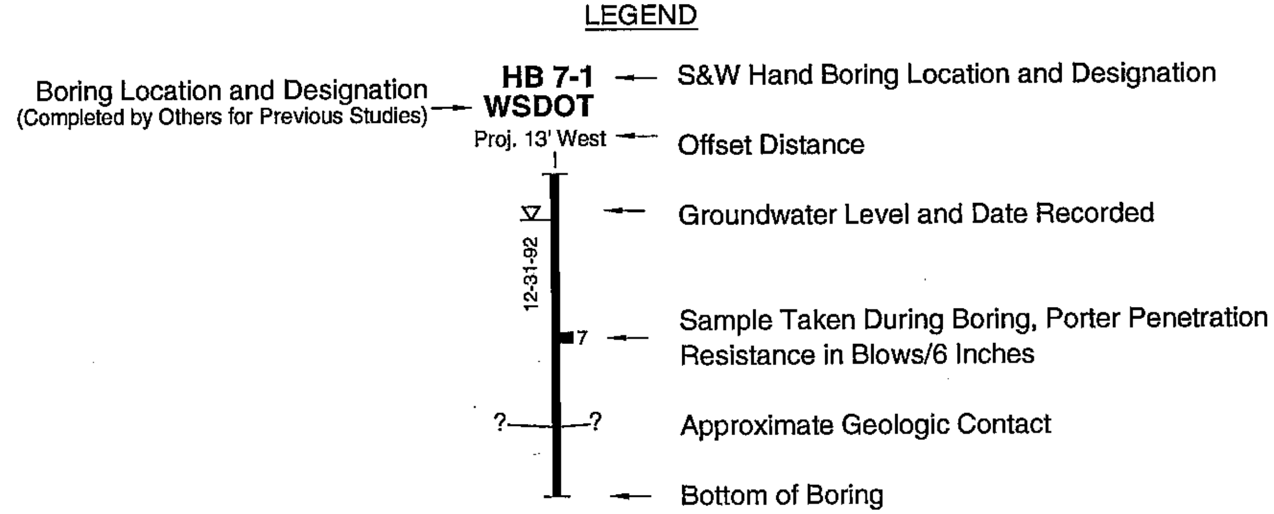
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 10

FIG. 10



LEGEND



NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.
2. Profile as drawn has a 5x vertical exaggeration.
3. Refer to individual boring logs in Appendix A for details of each boring.

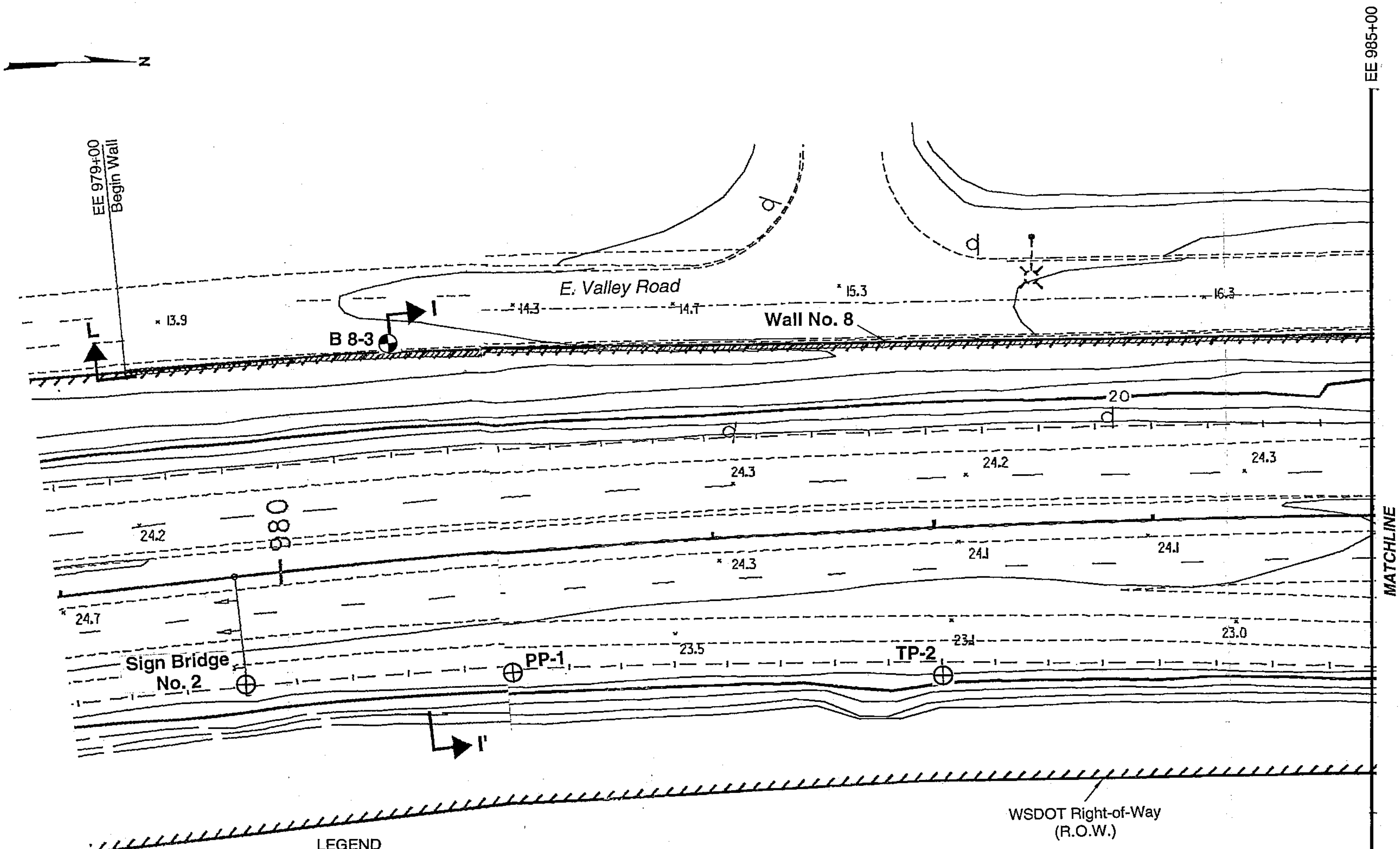
SR 167 H.O.V. Lanes
Renton, Washington

SUBSURFACE PROFILE H-H'
WALL NO. 7
L 1075+60 to 1083+50

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 11



LEGEND

B 8-1 — S&W Boring Designation and Approximate Location Completed for this Study

TP-3, PP-1 — Boring Designation and Approximate Location Completed by Others for Previous Studies

Proposed Retaining Wall

L — Generalized Subsurface Profile

0 50 100

Scale in Feet

NOTES

1. Base map provided by WSDOT.

2. Contour intervals are 2 feet.

SR 167 H.O.V. Lanes
Renton, Washington

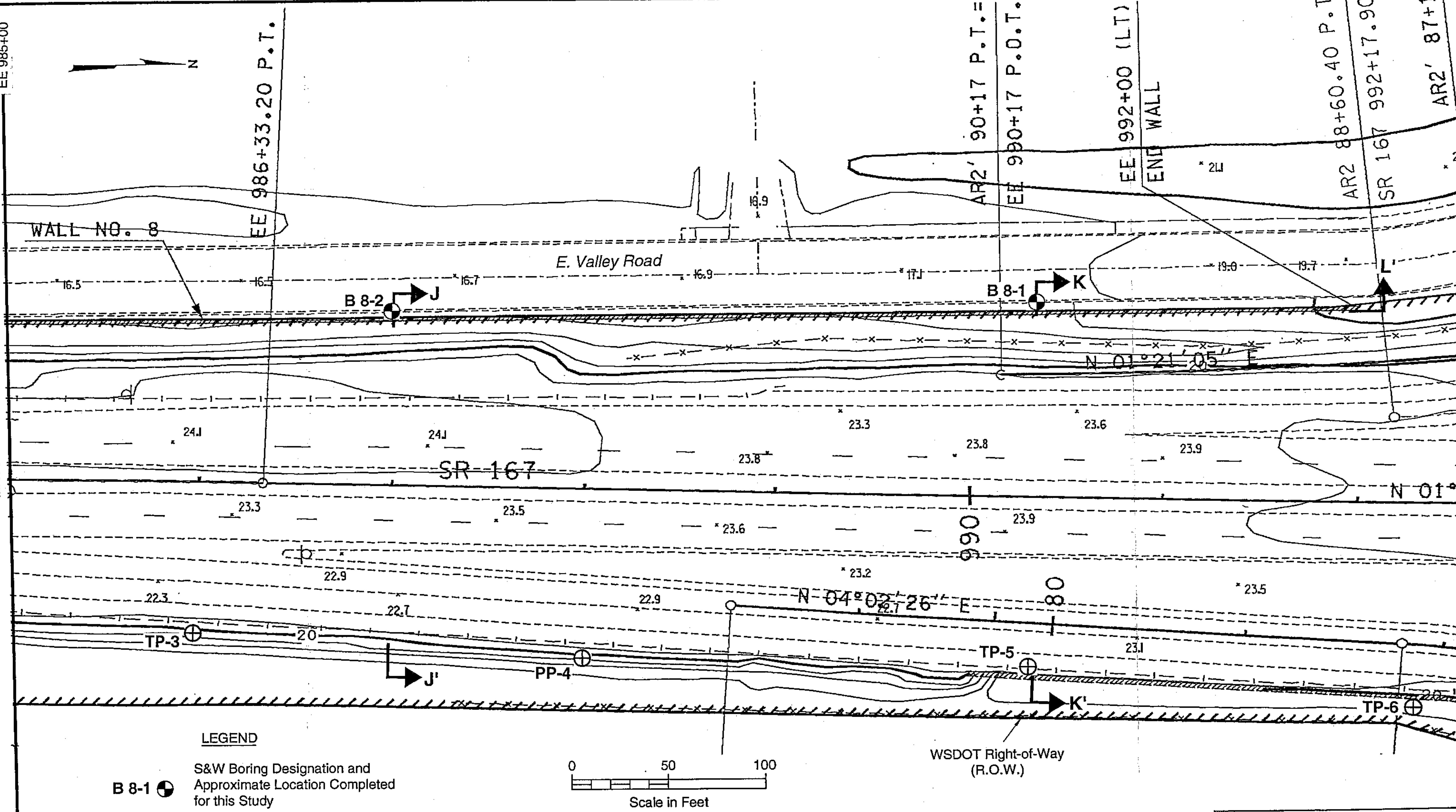
SITE AND EXPLORATION PLAN
WALL NO. 8

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 12

Sheet 1 of 2



LEGEND

B 8-1 — S&W Boring Designation and Approximate Location Completed for this Study

TP-3, PP-1 — Boring Designation and Approximate Location Completed by Others for Previous Studies

Proposed Retaining Wall

L — Generalized Subsurface Profile

0 50 100
Scale in Feet

NOTES

1. Base map provided by WSDOT.
2. Contour intervals are 2 feet.

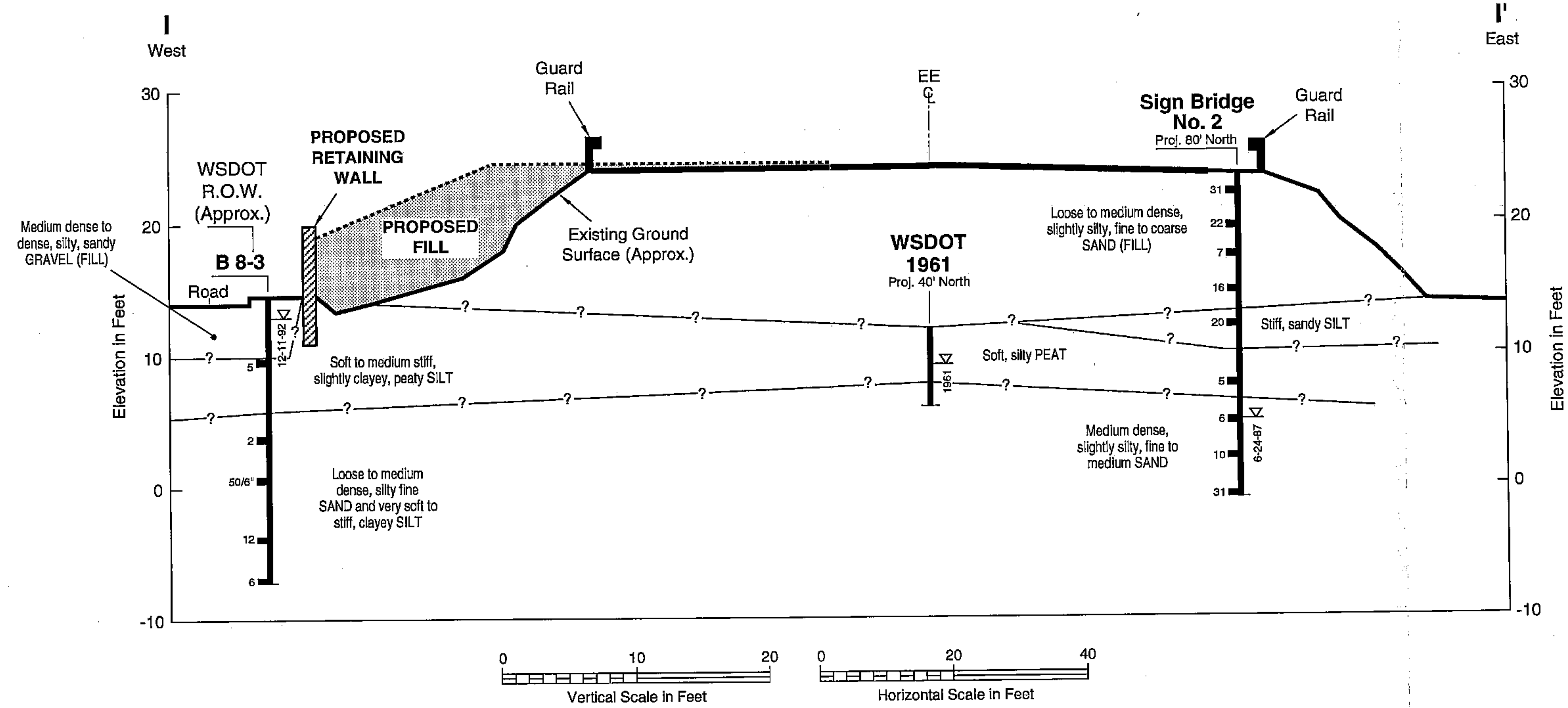
SR 167 H.O.V. Lanes
Renton, Washington

SITE AND EXPLORATION PLAN
WALL NO. 8

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 12



LEGEND

B 8-3 — S&W Boring Location and Designation
TP-3 — Boring Location and Designation Completed by Others for Previous Studies
Proj. 7' East — Offset Distance

12-18-92 — Groundwater Level and Date Recorded

7 — Sample Taken During Boring, Standard Penetration Resistance in Blows/Foot or Blows/Inches Driven

?——? Approximate Geologic Contact

Bottom of Boring

NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.

2. Profile as drawn has a 2x vertical exaggeration.

3. Refer to individual boring logs in Appendix A for details of each boring.

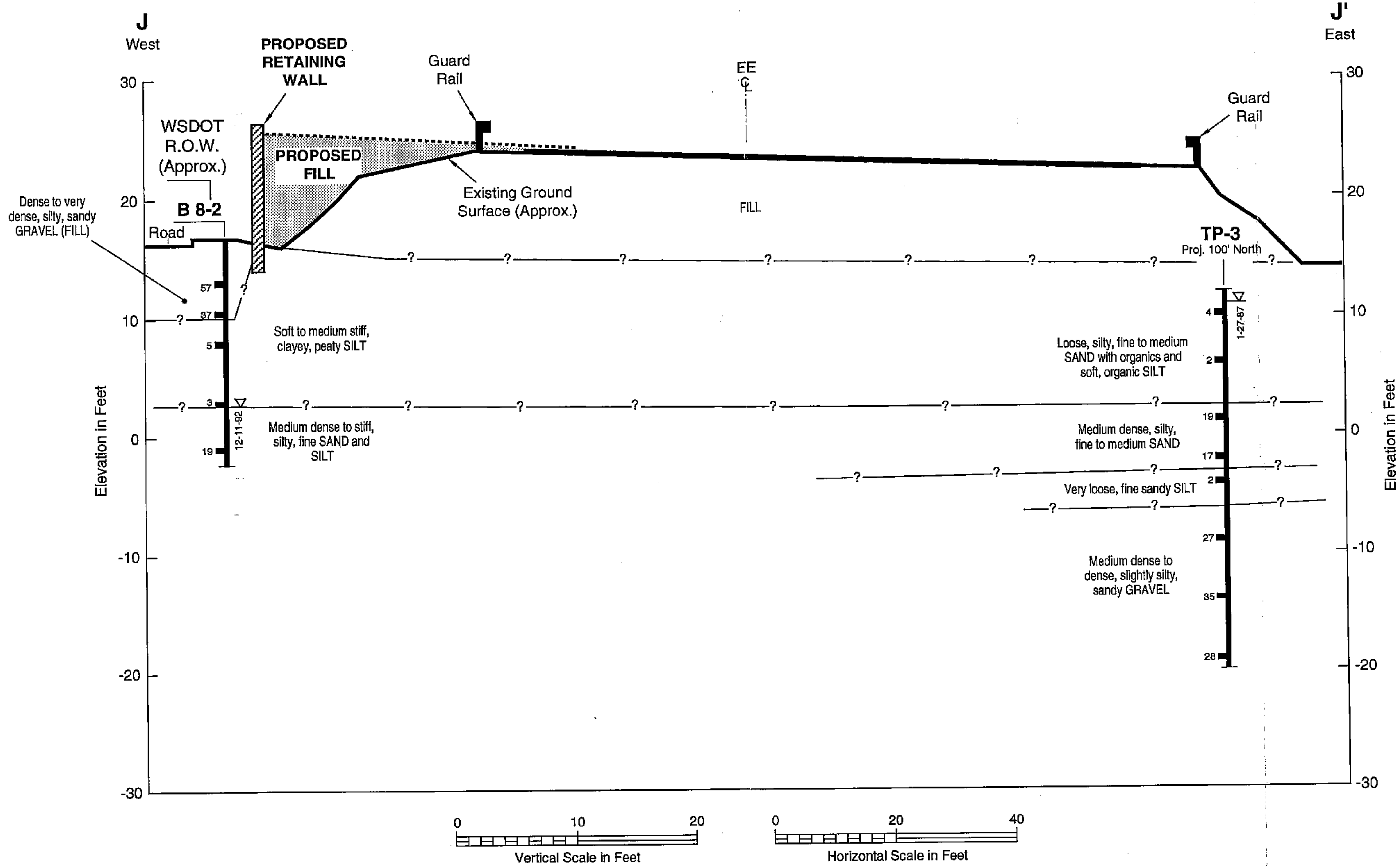
SR 167 H.O.V. Lanes
Renton, Washington

SUBSURFACE PROFILE I-I'
WALL NO. 8
EE 980+60

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 13



LEGEND

B 8-3 — S&W Boring Location and Designation

TP-3 — Boring Location and Designation Completed by Others for Previous Studies

Proj. 7' East — Offset Distance

12-18-92 — Groundwater Level and Date Recorded

7 — Sample Taken During Boring, Standard Penetration Resistance in Blows/Foot or Blows/Inches Driven

?——? Approximate Geologic Contact

Bottom of Boring

NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.
2. Profile as drawn has a 2x vertical exaggeration.
3. Refer to individual boring logs in Appendix A for details of each boring.

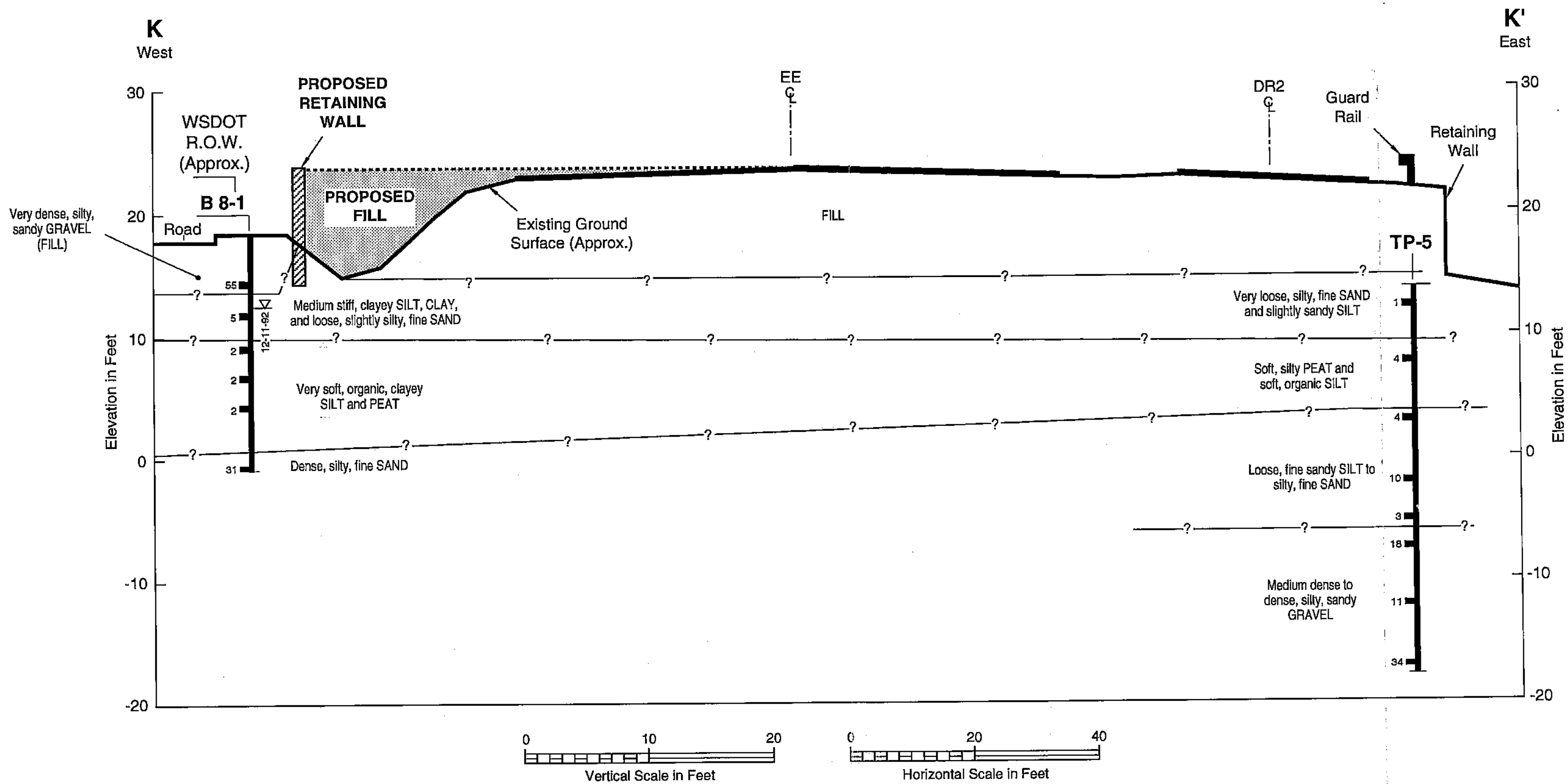
SR 167 H.O.V. Lanes
Renton, Washington

SUBSURFACE PROFILE J-J'
WALL NO. 8
EE 987+00

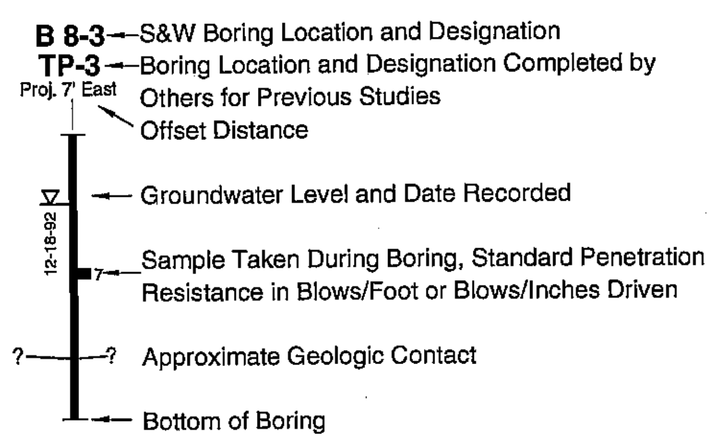
January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 14



LEGEND



NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.
2. Profile as drawn has a 2x vertical exaggeration.
3. Refer to individual boring logs in Appendix A for details of each boring.

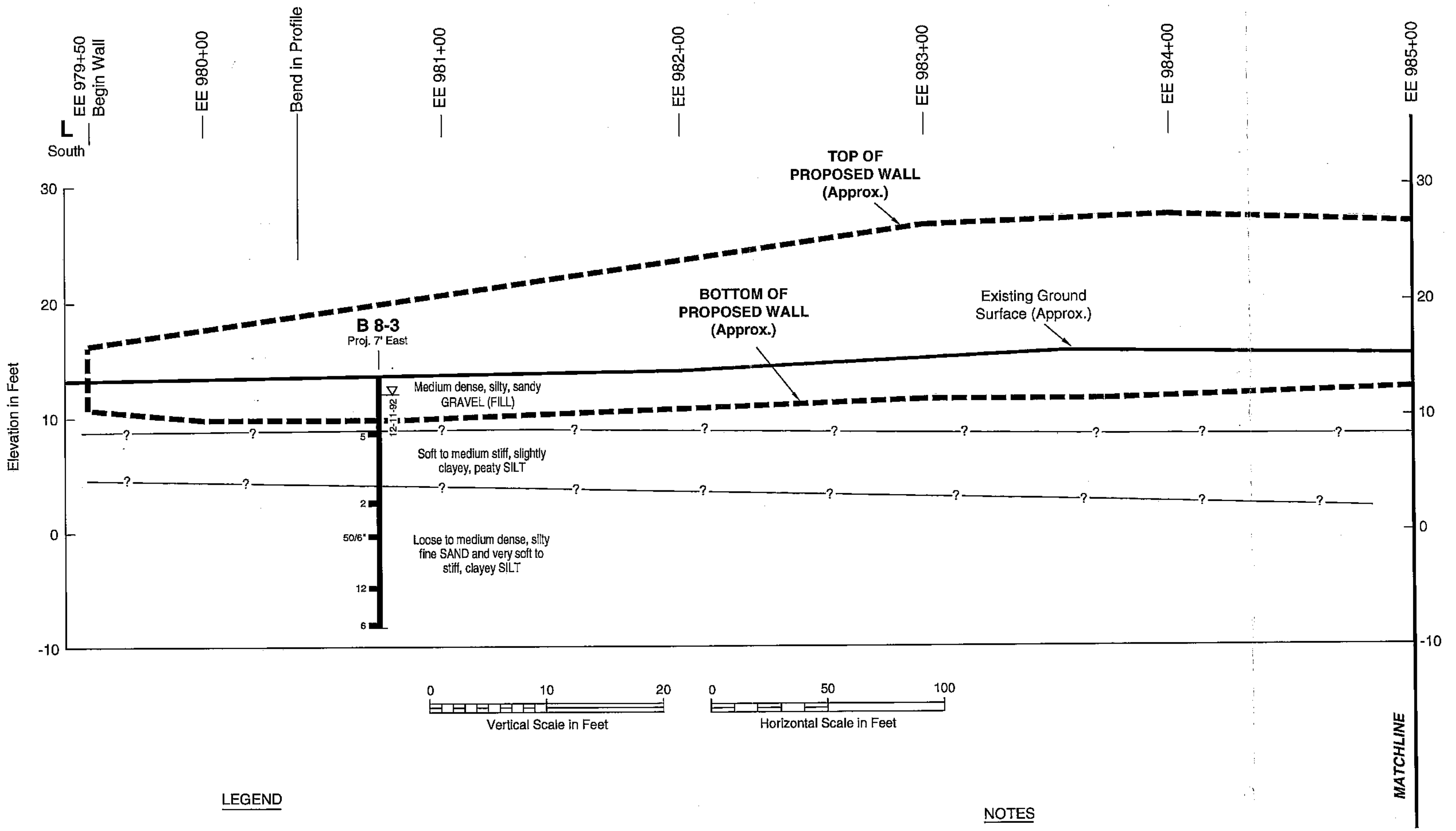
SR 167 H.O.V. Lanes
Renton, Washington

SUBSURFACE PROFILE K-K'
WALL NO. 8
EE 990+30

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 15



LEGEND

B 8-3 — S&W Boring Location and Designation

Proj. 7' East — Offset Distance

12-18-92 — Groundwater Level and Date Recorded

7 — Sample Taken During Boring, Standard Penetration Resistance in Blows/Foot or Blows/Inches Driven

?——? Approximate Geologic Contact

Bottom of Boring

NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.
2. Profile as drawn has a 5x vertical exaggeration.
3. Refer to individual boring logs in Appendix A for details of each boring.

SR 167 H.O.V. Lanes
Renton, Washington

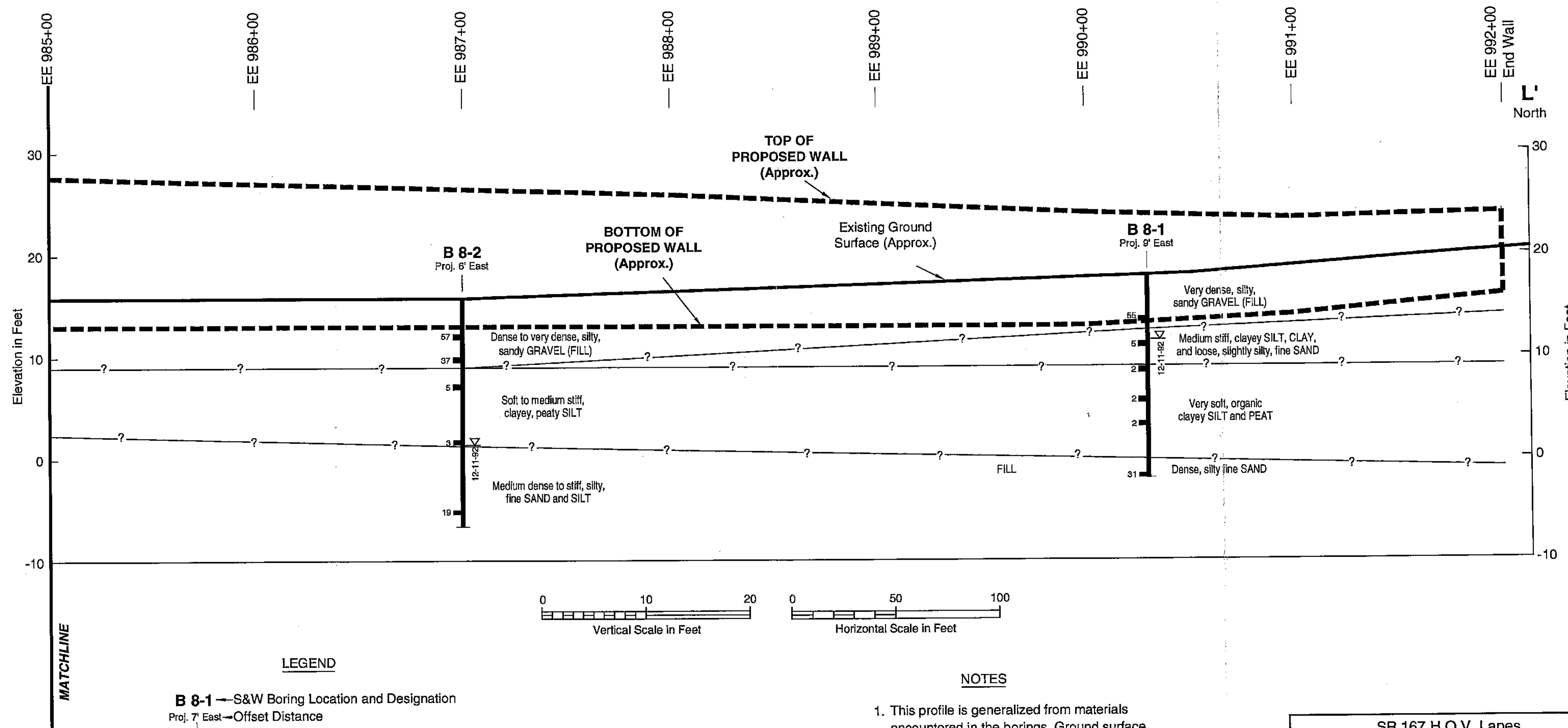
SUBSURFACE PROFILE L-L'
WALL NO. 8
EE 979+40 to 992+10

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 16

Sheet 1 of 2



LEGEND

B 8-1 — S&W Boring Location and Designation
Proj. 7' East — Offset Distance

Groundwater Level and Date Recorded

Sample Taken During Boring, Standard Penetration Resistance in Blows/Foot or Blows/Inches Driven

Approximate Geologic Contact

Bottom of Boring

NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.
2. Profile as drawn has a 5x vertical exaggeration.
3. Refer to individual boring logs in Appendix A for details of each boring.

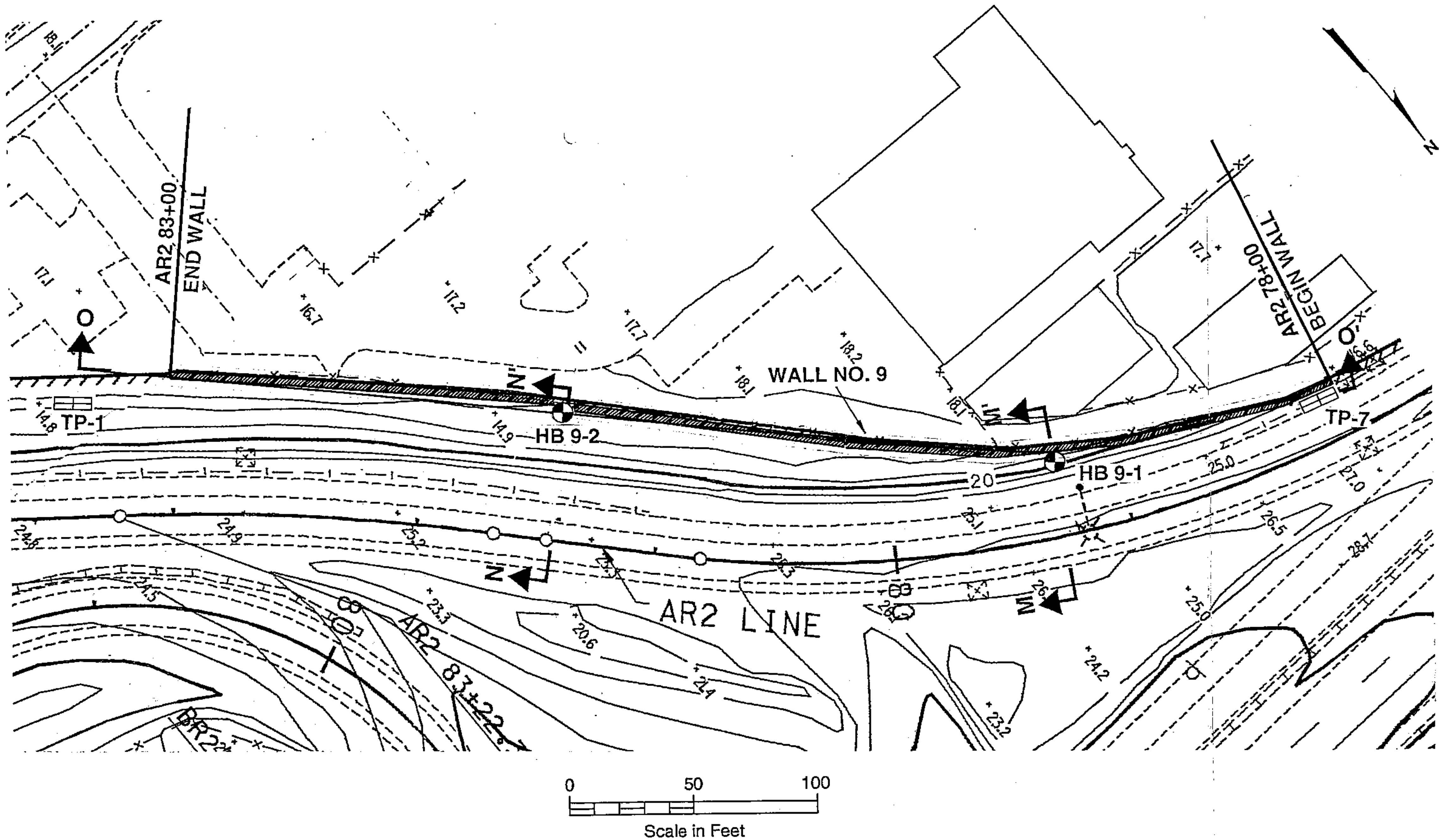
SR 167 H.O.V. Lanes
Renton, Washington

SUBSURFACE PROFILE L-L'
WALL NO. 8
EE 979+40 to 992+10

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 16



LEGEND

HB 9-2 — S&W Hand Boring Designation and Approximate Location Completed for this Study

TP-1 — Test Pit Designation and Approximate Location Completed by Others for Previous Studies

Proposed Retaining Wall

M — Generalized Subsurface Profile

NOTES

1. Base map provided by WSDOT.
2. Contour intervals are two feet.

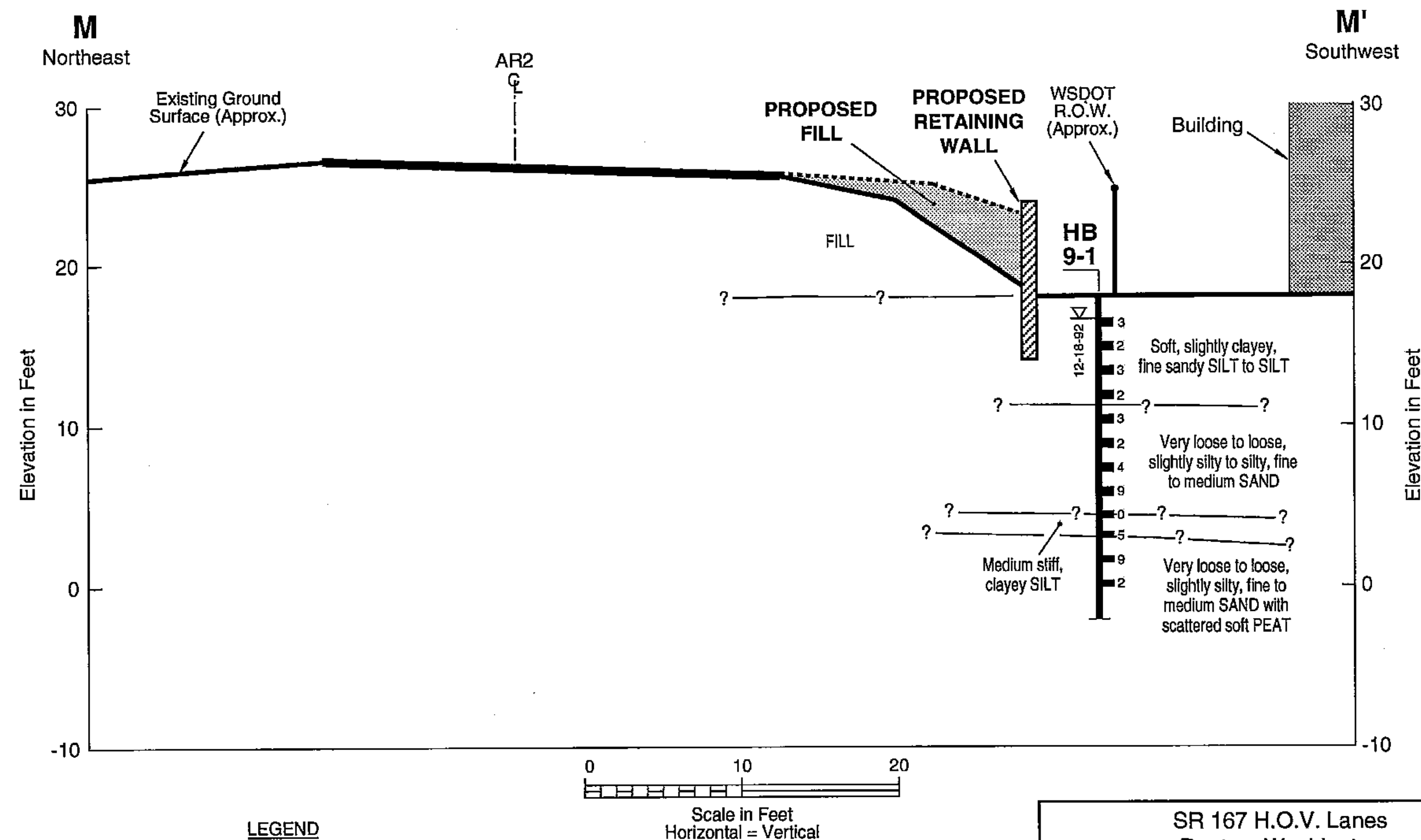
SR167 H.O.V. Lanes
Renton, Washington

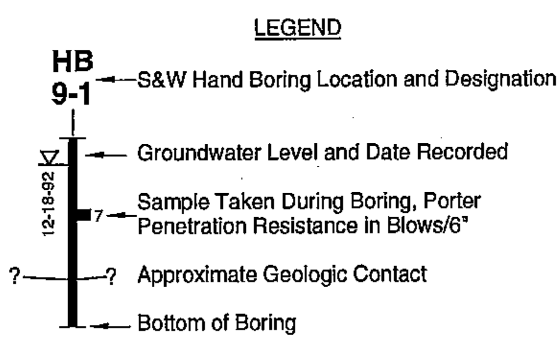
SITE AND EXPLORATION PLAN WALL NO. 9

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 17





NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base map provided by WSDOT. Variations between the profile and the actual conditions may exist.
2. Refer to individual boring logs in Appendix A for details of each boring.

SR 167 H.O.V. Lanes
Renton, Washington

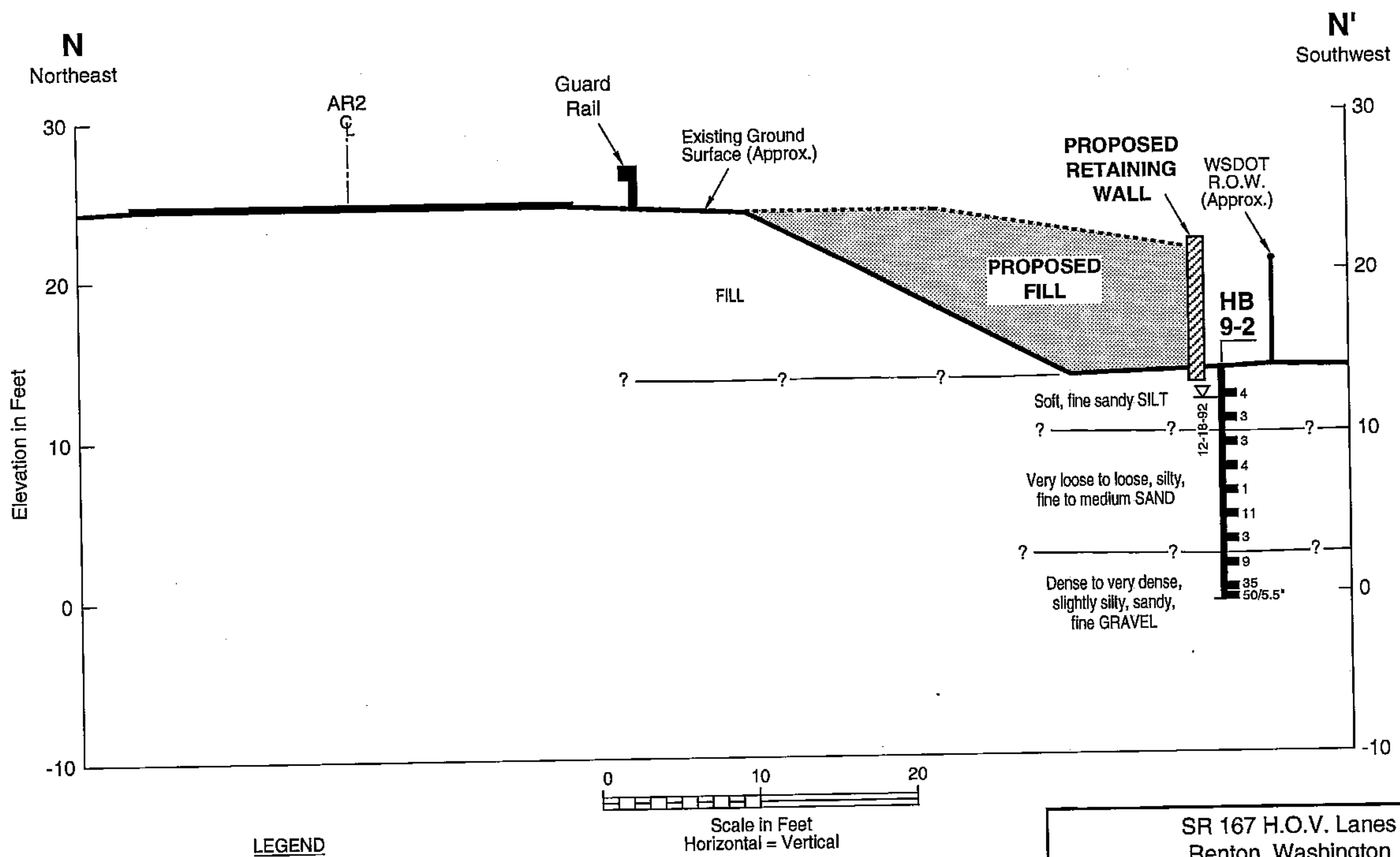
SUBSURFACE PROFILE M-M'
RETAINING WALL NO. 9
AR2 79+27

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 18

FIG. 18



LEGEND

- HB 9-2 — S&W Hand Boring Location and Designation
- 12-18-92 — Groundwater Level and Date Recorded
- 7 or 50/4" — Sample Taken During Boring, Porter Penetration Resistance in Blows/6" or Blows/Inches Driven
- ?—? — Approximate Geologic Contact
- Bottom of Boring

NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.
2. Refer to individual boring logs in Appendix A for details of each boring.

SR 167 H.O.V. Lanes
Renton, Washington

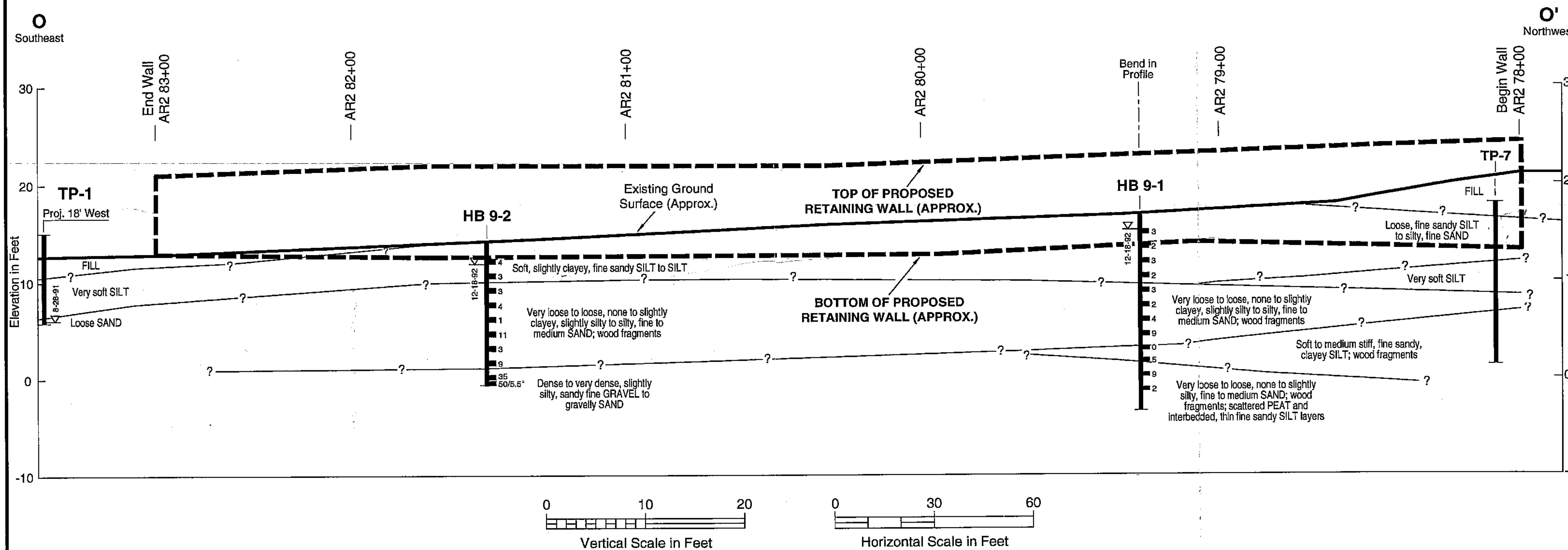
SUBSURFACE PROFILE N-N'
WALL NO. 9
AR2 81+43

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 19

FIG. 19



LEGEND

Boring Location and Designation (Completed by Others for Previous Studies) → **HB 9-1** ← S&W Hand Boring Location and Designation

WSDOT

Proj. 13' West ← Offset Distance

12-31-92 ← Groundwater Level and Date Recorded

7 or 50/4" ← Sample Taken During Boring, Porter Penetration Resistance in Blows/6 Inches

?—? Approximate Geologic Contact

← Bottom of Boring

NOTES

1. This profile is generalized from materials encountered in the borings. Ground surface elevations derived from base maps provided by WSDOT. Variations between the profile and the actual conditions may exist.
2. Profile as drawn has a 3x vertical exaggeration.
3. Refer to individual boring and test pit logs in Appendix A for details of each boring.

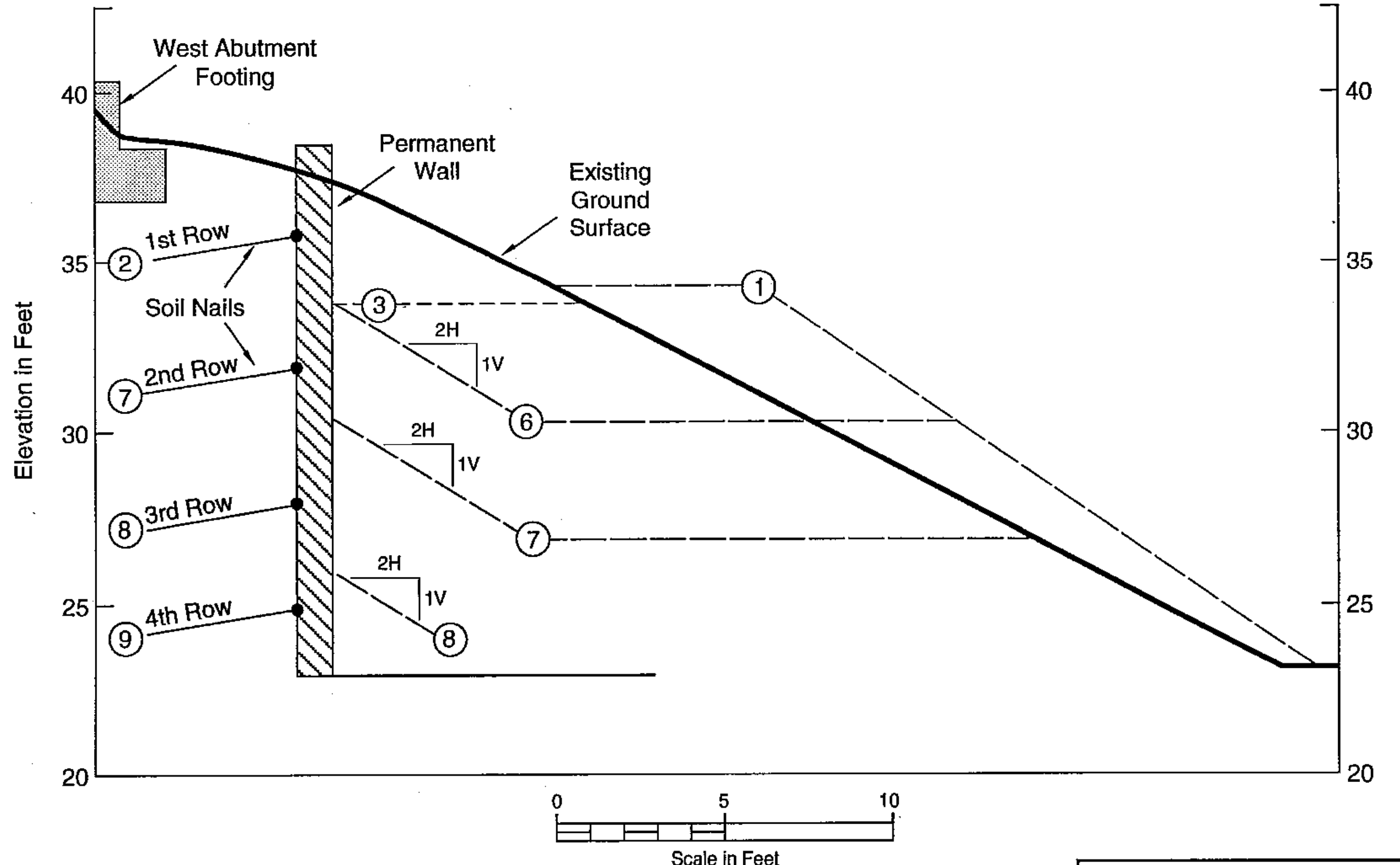
SR 167 H.O.V. Lanes
Renton, Washington

SUBSURFACE PROFILE O-O'
WALL NO. 9
AR2 77+85 TO 83+40

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 20



NOTES

The following construction sequence applies to Wall 6 between stations 1054+55 and 1055+64:

1. Construct berm to install first nail row.
2. Install Nails
3. Excavate alternate slots along the wall at maximum length of 12 feet.
4. Place drainage mat, shotcrete and prestress nails.
5. Excavate remaining slots, place drainage mat, shotcrete and prestress nails.
6. Excavate 2H:1V slope from base of shotcrete to next berm level. Top of berm should be no lower than 2 feet below the last row of completed nails.
7. Repeat 2, 3, 4, 5 and 6.
8. Repeat 2, 3, 4, 5 and 6.
9. Repeat 2, 3, 4 and 5.

SR 167 H.O.V. Lanes
Renton, Washington

RECOMMENDED CONSTRUCTION SEQUENCE WALL 6 AT BRIDGE

February 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 21

FIG. 21

APPENDIX A

FIELD EXPLORATIONS

APPENDIX A

FIELD EXPLORATIONS

Appendix A contains logs of subsurface explorations in the vicinity of the proposed Walls 6, 7, 8, and 9. These explorations include 15 borings and 3 test pits which were advanced for this study and 13 borings and 3 test pits previously advanced for other WSDOT projects. The exploration logs are arranged generally from south to north and by wall location. Table A-1 lists the explorations, wall , station, offset, elevation, depth, and Figure number for each of the explorations. The location of the subsurface locations are shown on the site plans, (Figures 2, 7, 12 and 17) in the main body of the report.

Of the 15 borings performed specifically for this study, 10 were advanced using hand-boring equipment: 2 hand borings were advanced in the vicinity of wall 9, and 8 were advanced in the vicinity of wall 7. Hand borings were located in areas inaccessible to truck-mounted drilling equipment. The remaining 5 borings were advanced using a truck-mounted, hollow-stem auger drill rig under subcontract to Environmental Drilling Inc. of Snohomish, Washington. Three of the truck-mounted borings were advanced in the vicinity of wall 8, and 2 were advanced near wall 6. The dates that the borings were completed are indicated on the boring logs.

Three test pits were excavated for this study in the vicinity of wall 6 by WSDOT maintenance personnel on December 28, 1992. They were accomplished to evaluate the subsurface conditions near the face of wall 6 and evaluate the stand-up characteristics of these soils for a proposed soil-nail wall.

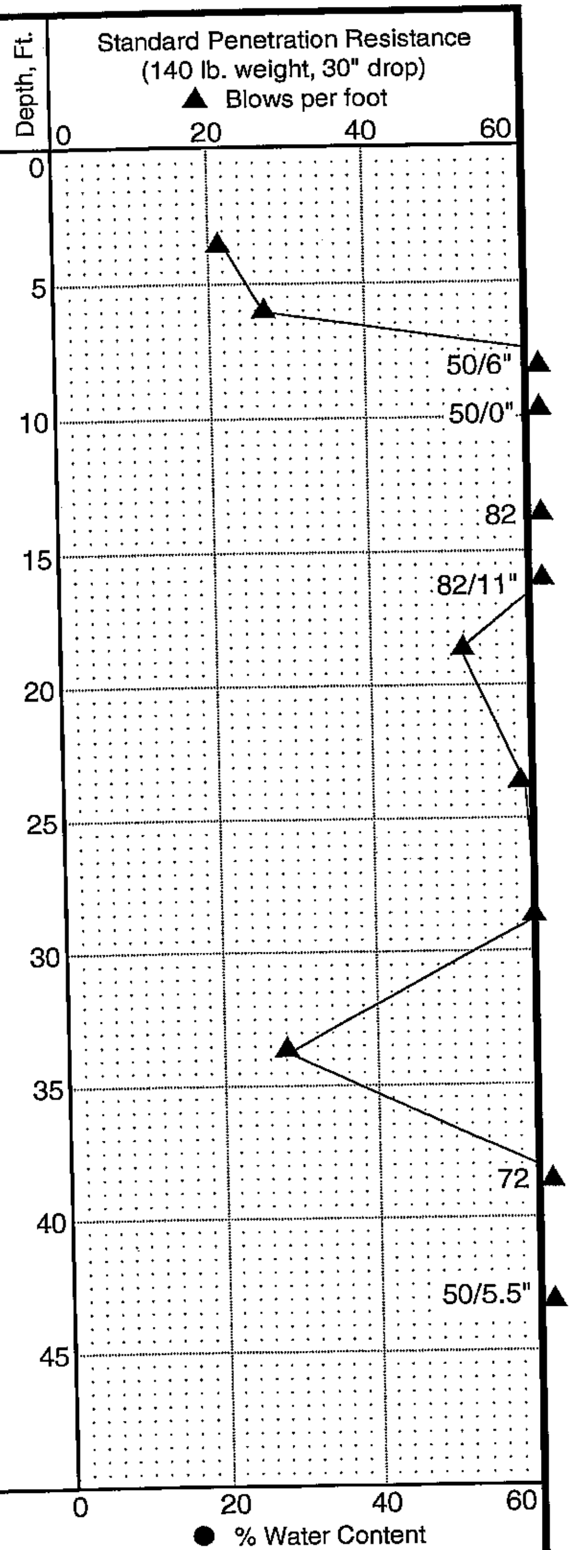
All explorations conducted specifically for this study were conducted in the presence of an experienced engineering geologist from Shannon & Wilson, Inc., who logged the materials encountered in the explorations and collected representative samples. These samples were returned to Shannon & Wilson's laboratory for further visual classification. Index testing, consisting of water content and Atterberg limit determinations were performed on selected samples. The results of the visual classifications and index testing are summarized on the exploration logs and are discussed in greater detail in Appendix B.

TABLE A-1

SR 167 H.O.V. LANE BORINGS AND TEST PITS

WALL	BORING	S&W	LINE	STATION	OFFSET	ELEVATION (feet)	DEPTH (feet)	FIGURE
6	B 6-1	YES	L	1056+35	157' Lt.	42.0	43.5	A-1
6	B 6-2	YES	L	1053+32	130' Lt.	42.0	30.0	A-2
6	TP 6-1	YES	L	1055+85	80' Lt.	33.0	9.0	A-3
6	TP 6-2	YES	L	1054+45	80' Lt.	31.0	8.0	A-4
6	TP 6-3	YES	L	1053+05	75' Lt.	33.0	9.0	A-5
6	WSDOT'61		L	1050+00	0	26.0	16.0	A-6
6	H-2		L	1054+75	85' Lt.	45.0	47.0	A-7
6	TP-5		L	1053+55	65' Lt.	26.5	7.0	A-8
7	HB 7-1	YES	L	1082+05	75' Rt.	15.0	9.0	A-9
7	HB 7-2	YES	L	1079+00	73' Rt.	15.5	10.5	A-10
7	HB 7-3	YES	L	1076+10	78' Rt.	20.0	15.0	A-11
7	HB 7-4	YES	L	1076+10	90' Rt.	19.5	12.0	A-12
7	HB 7-5	YES	L	1079+00	95' Rt.	15.5	9.0	A-13
7	HB 7-6	YES	L	1082+05	95' Rt.	14.0	6.5	A-14
7	HB 7-7	YES	L	1071+10	85' Rt.	23.0	12.0	A-15
7	HB 7-8	YES	L	1068+75	90' Rt.	24.0	7.5	A-16
7	WSDOT '61		L	1076+75	50' Lt.	15.5	9.5	A-17
7	WSDOT '61		L	1079+80	0	11.5	4.0	A-17
7	WSDOT '61		L	1080+80	50' Lt.	11.0	5.5	A-17
7	WSDOT '61		L	1082+90	50' Rt.	11.0	6.0	A-17
8	B 8-1	YES	EE	990+00	98' Lt.	18.0	19.0	A-18
8	B 8-2	YES	EE	987+00	90' Lt.	17.0	19.0	A-19
8	B 8-3	YES	EE	980+60	96 Lt.	14.0	21.5	A-20
8	Sign Brdg 2		EE	979+80	48' Rt.	23.5	24.5	A-21
8	PP 1		EE	981+00	53' Rt.	19.0	7.0	A-22
8	TP 2		EE	983+00	65' Rt.	15.5	36.5	A-23
8	TP 3		EE	986+00	80' Rt.	11.8	32.0	A-24
8	PP 4		EE	988+00	87' Rt.	12.5	16.5	A-25
8	TP 5		EE	990+30	87' Rt.	14.0	32.0	A-26
8	TP 6		EE	992+25	105' Rt.	16.0	29.0	A-27
9	HB 9-1	YES	AR2	79+25	35' Rt.	17.0	20	A-28
9	HB 9-2	YES	AR2	81+44	57' Rt.	14.0	14.5	A-29
9	TP-7		AR2	78+10	18' Rt.	16.0	15	A-30
9	TP-1		AR2	83+40	47' Rt.	15	9	A-31

SOIL DESCRIPTION Surface Elevation: Approx. 42 Feet	Depth, Ft.	Samples	Ground Water
Medium dense, brown, slightly gravelly, silty, fine SAND to fine sandy SILT; scattered orange iron oxide staining; moist	0	1, 2	
Very dense, brown, slightly gravelly, slightly silty to silty SAND; moist	7	3, 4	
Very dense, gray, slightly fine sandy SILT; moist	11	5, 6	
Very dense, brown to gray, silty, fine SAND; trace of gravel; scattered lenses (up to 4" thick) of silt; sand is wet, silt is moist	16	7, 8, 9	During Drilling ▽
Medium dense, gray, fine SAND; wet	31	10	
Very dense, gray, silty, fine SAND to slightly fine sandy SILT; sand is wet, silt is moist	37	11, 12	
BOTTOM OF BORING COMPLETED 12-28-92	43.5		



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- * Sample not recovered
- Atterberg limits: Liquid limit, Natural water content, Plastic limit
- Impervious seal
- ▽ Water level
- Piezometer tip
- P Sample pushed

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

SR 167 H.O.V. Lanes
Renton, Washington

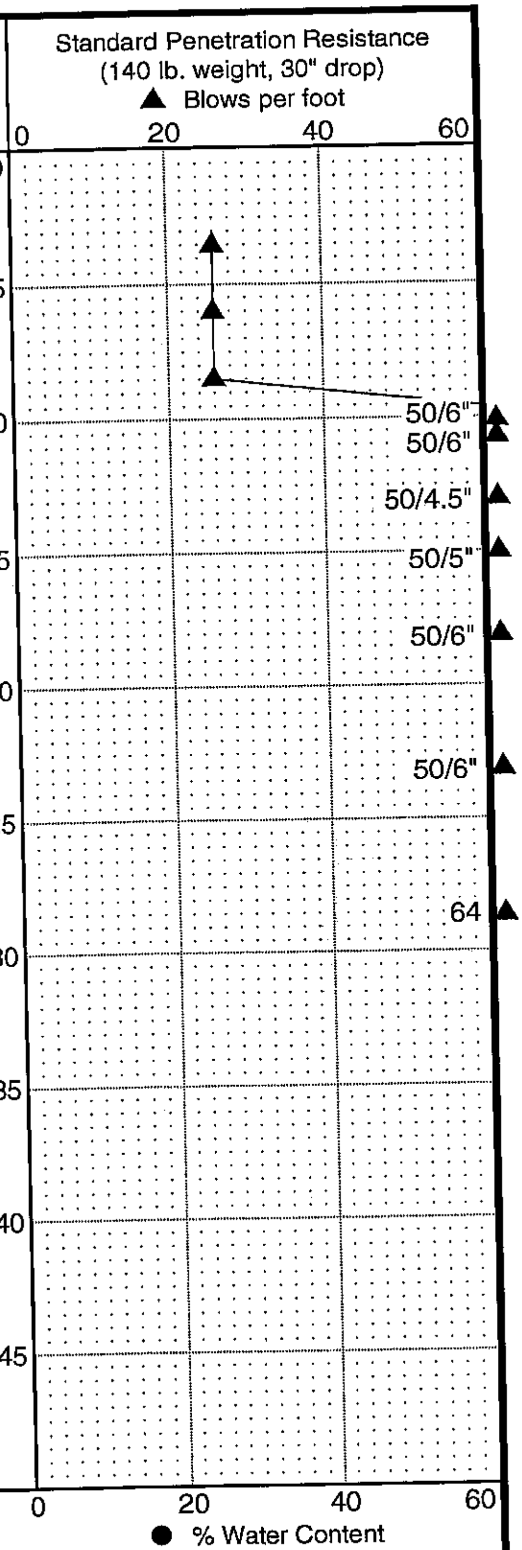
LOG OF BORING B 6-1
WALL NO. 6
STA 1056+35, 157' Lt. L

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1

SOIL DESCRIPTION Surface Elevation: Approx. 42 Feet	Depth, Ft.	Samples	Ground Water
Medium dense, gray-brown SILT to fine sandy SILT; with scattered orange iron-oxide staining; moist	0	1	
Medium dense, brown, fine to medium SAND; trace of fine gravel; scattered lenses (up to 4" thick) of fine sandy silt; moist	4.5	2, 3	
Very dense, tan to dark brown, slightly silty, slightly sandy GRAVEL to silty, gravelly SAND; moist	10.5	4, 5, 6	
Very dense, light gray, slightly silty to silty, fine SAND moist	16.5	7	
Very dense, gray-brown, silty SAND with a trace of fine gravel; moist	20	8	During Drilling ▽
Very dense, gray, silty, gravelly SAND; boulder (?) at 30'; wet	25	9	
BOTTOM OF BORING COMPLETED 12-28-92	30		



NOTE

Boulder (?) at 30' beyond which the auger could not advance.

LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- * Sample not recovered
- Impervious seal
- ▽ Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

SR 167 H.O.V. Lanes
Renton, Washington

LOG OF BORING B 6-2
WALL NO. 6
STA 1053+32, 130' Lt. L

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-2

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP 6-1

JOB NO: W-6391-03 DATE: 12-28-92 LOCATION: STA 1055+85, 80' Lt. L

PROJECT: SR 167 H.O.V. Lane Walls

Sketch of West Pit Side

Surface Elevation: Approx. 33 Feet

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
			S-1	0
(1) Loose to medium dense, brown, slightly clayey, silty, fine to medium SAND; scattered gravel, fine to 1 inch roots; iron-oxide stain; moist			S-4	2
(2) Dense to very dense, mottled gray-brown, slightly clayey, silty SAND to clayey, sandy SILT; scattered gravel and cobbles; iron-oxide stain; charcoal; moist (Weathered TILL)			S-3	
			S-2	4
(3) Dense to very dense, gray, clayey, silty SAND to clayey, sandy SILT; scattered gravel; granitic and siltstone clasts and sand stringers; moist (TILL)				6
(4) Medium dense to dense, brown, slightly silty, sandy GRAVEL to gravelly SAND; scattered cobbles; iron-oxide stain; wood fragments; charcoal; moist			S-5	8
(5) Dense, gray, silty, fine SAND; moist	♀			
(6) Medium dense to dense, gray, slightly silty, sandy GRAVEL to gravelly SAND; moist to wet; seepage at toe				10
				12

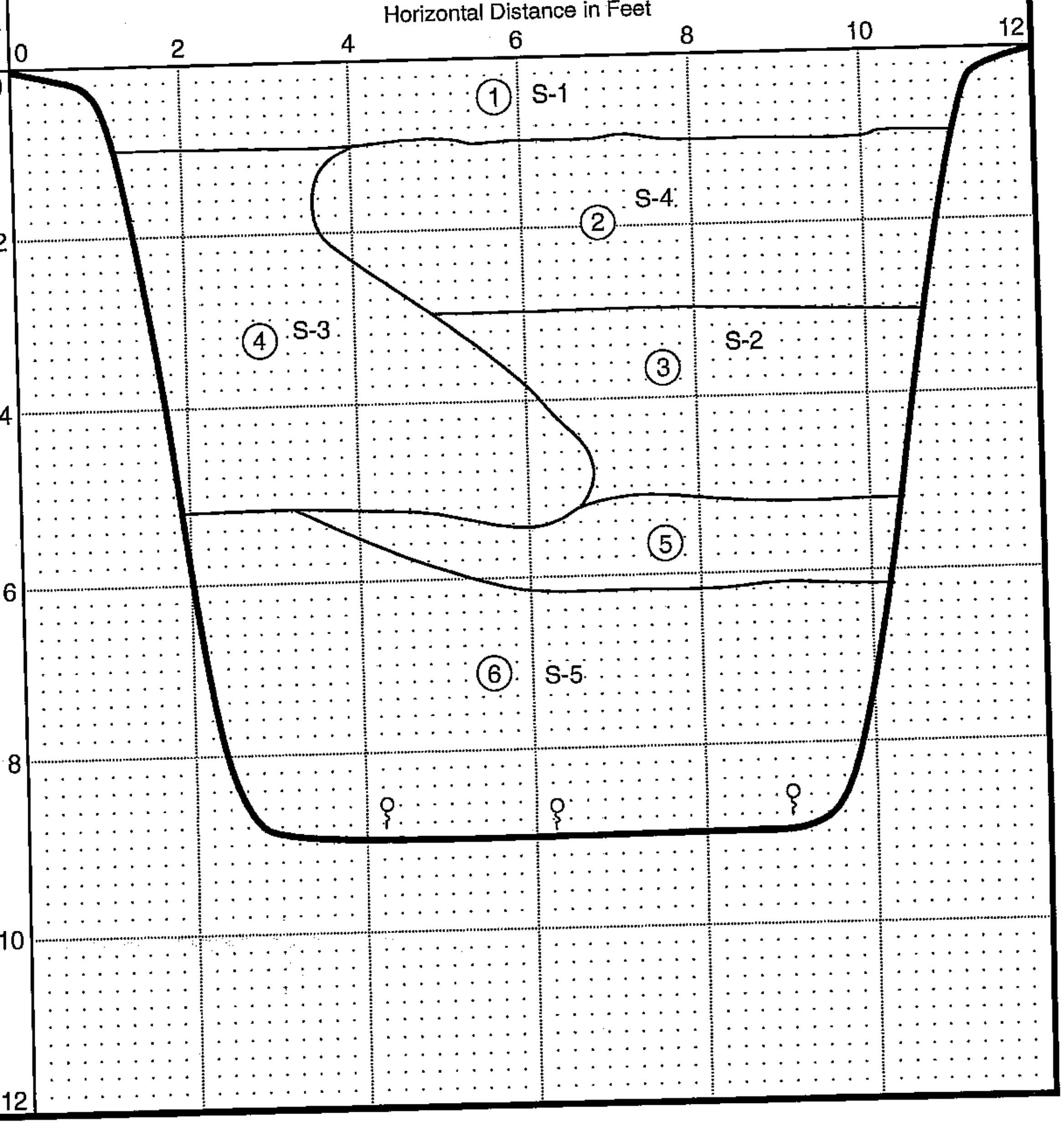


FIG. A-3

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP 6-2

JOB NO: W-6391-03 DATE: 12-28-92 LOCATION: STA 1054+45, 80' Lt. L

PROJECT: SR 167 H.O.V. Lane Walls

Sketch of South Pit Side Surface Elevation: Approx. 31 Feet

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
1 Loose to medium dense, brown to mottled gray-brown, slightly silty to silty, fine to medium SAND; scattered fine gravel; 1/2" roots; iron-oxide stain; moist			S-1	2
2 Dense to very dense, gray, fine sandy SILT; scattered fine gravel; wood fragments; charcoal; moist (Weathered TILL)			S-2	4
3 Medium dense, gray, slightly silty to silty, fine to medium SAND; moist			S-3	6
4 Very stiff to hard, gray, slightly silty to silty CLAY; moist			S-4	8

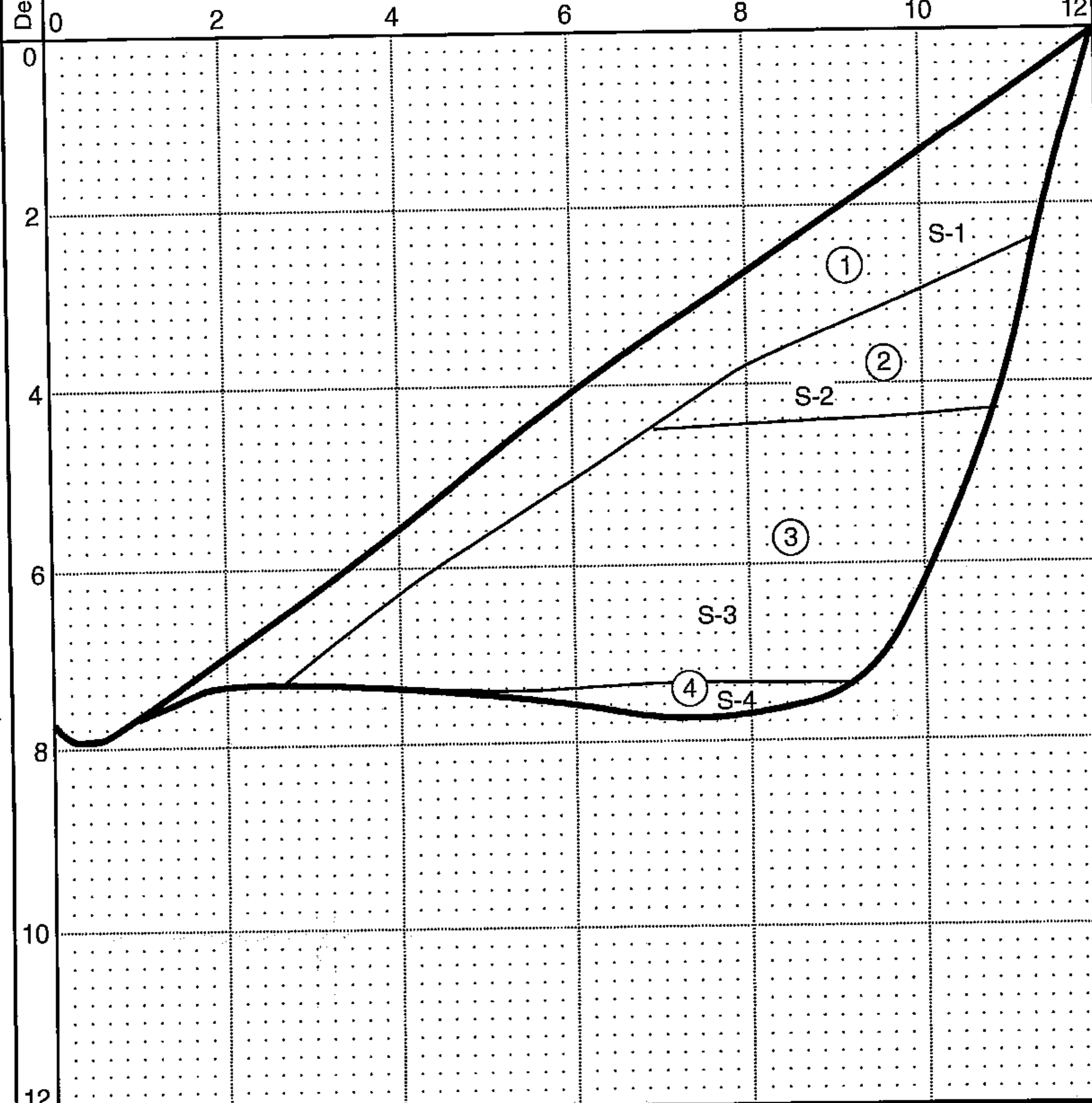


FIG. A-4

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP 6-3

JOB NO: W-6391-03 DATE: 12-28-92 LOCATION: STA 1053+05, 75' Lt. L

PROJECT: SR 167 H.O.V. Lane Walls

Sketch of West Pit Side

Surface Elevation: Approx. 33 Feet

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Loose to medium dense, brown, slightly clayey, silty fine SAND to fine sandy SILT; roots to 2"; wood fragments; moist			S-1	0
(2) Very stiff to dense, mottled gray-brown, fine sandy, clayey SILT to clayey, silty, fine SAND with clean, thin sand stringers; fine roots upper 2'; iron-oxide stain; moist			S-2	2, 4
(3) Medium dense, gray, clayey, silty, fine SAND to fine sandy SILT; scattered clean sand stringers; moist			S-3	6, 8, 10, 12

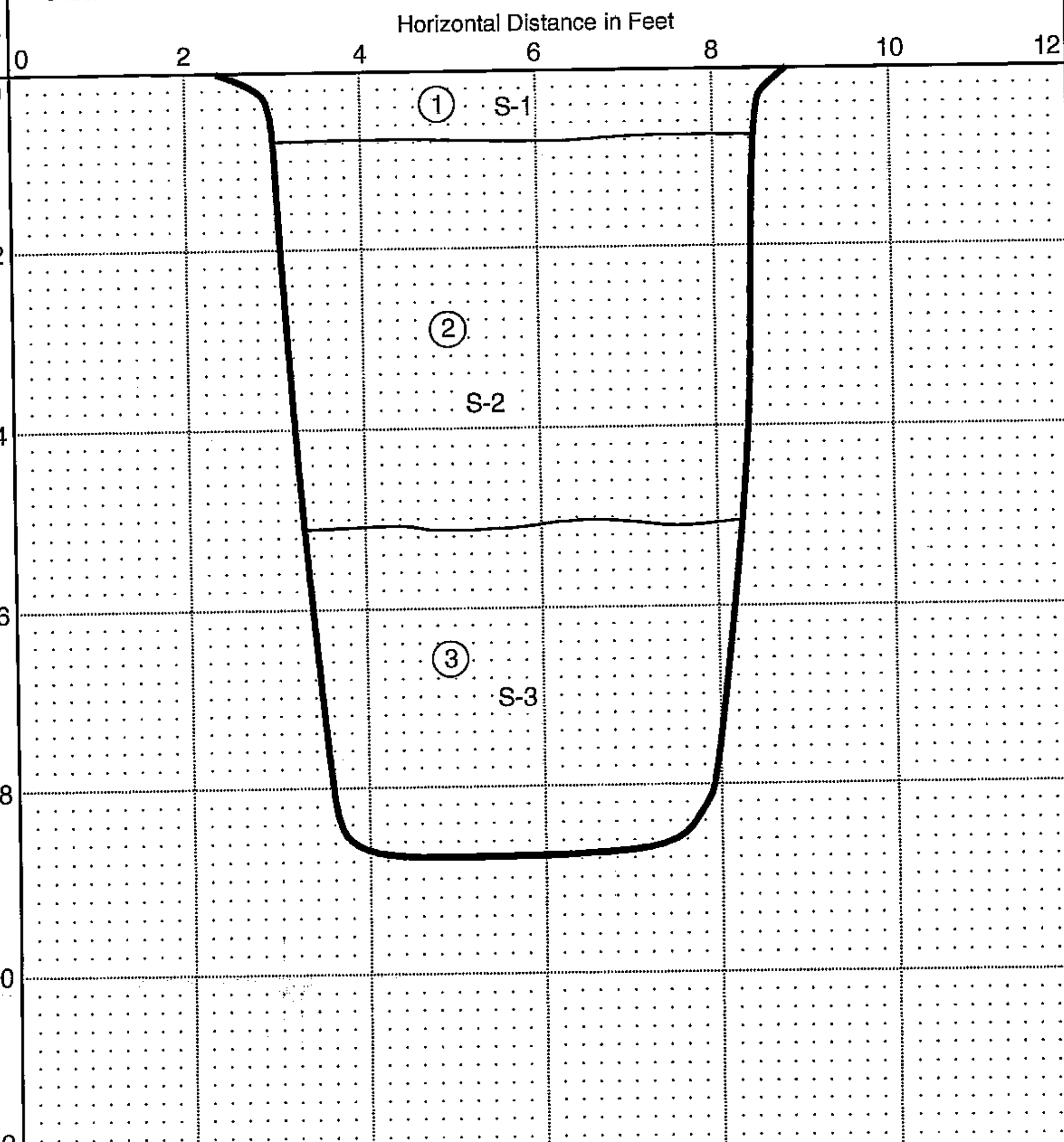
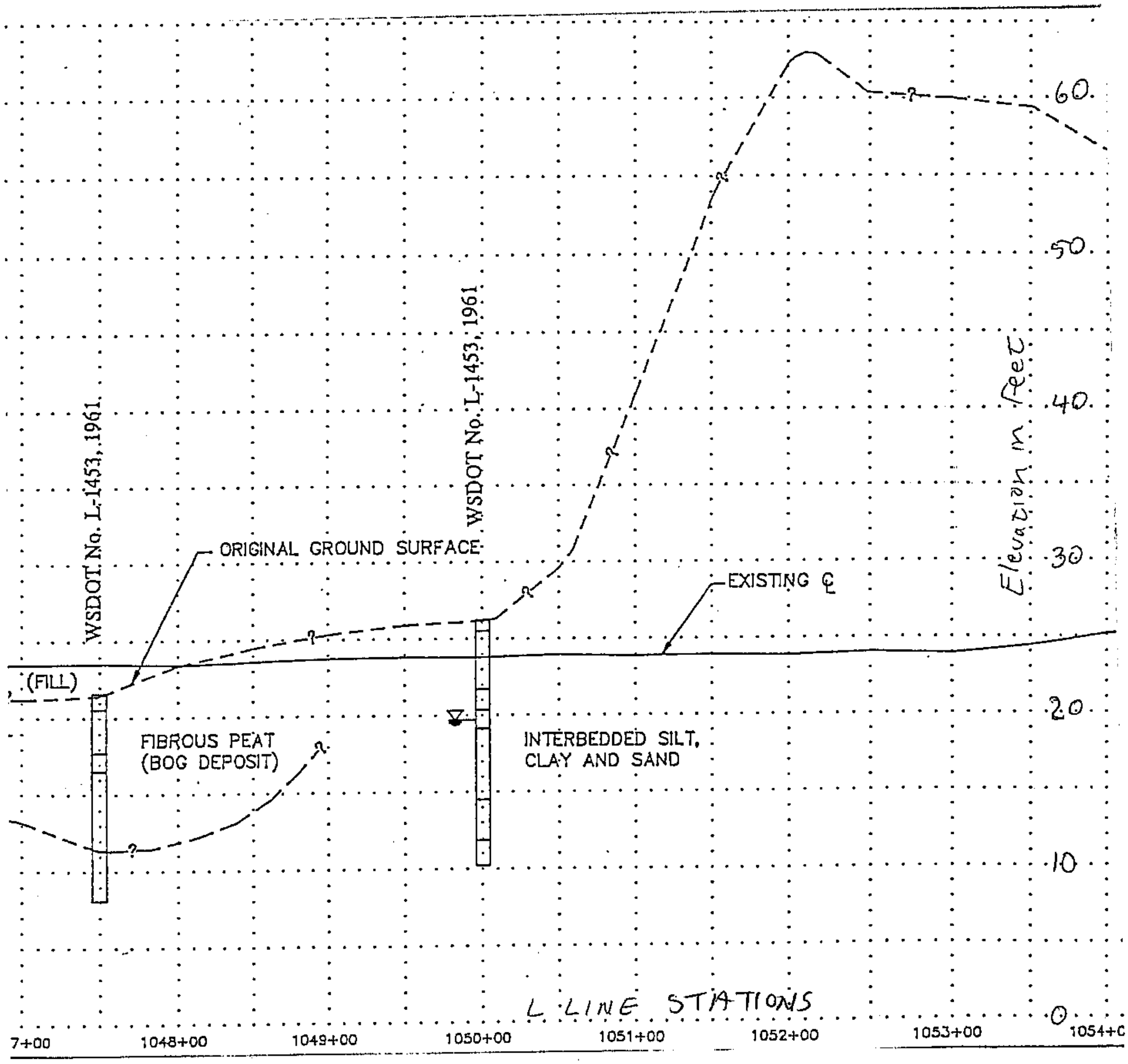


FIG. A-5



WSDOT No. L-1453, 1961
WSDOT No. L-1453, 1961
ORIGINAL GROUND SURFACE
EXISTING ℄
(FILL)
FIBROUS PEAT
(BOG DEPOSIT)
INTERBEDDED SILT,
CLAY AND SAND
Elevation in Feet
60
50
40
30
20
10
0
L LINE STATIONS
7+00
1048+00
1049+00
1050+00
1051+00
1052+00
1053+00
1054+0

Fig. A-6

Blows/Ft	Depth	
		Ground Elev. 45'
	2'	Brown all grades sand and gravel to 4"
9	5'	Stiff gray clay with silt and sand lens.
26	11'	Compact gray, silty all grades sand, occasional fine gravel.
	12'	
36	15'	Dense, gray silty all grades sand with fine gravel. Occasional piece of medium gravel.
38	17'	
39	19'	
19	25'	Slightly compact gray silty all grades sand and fine gravel.
15	30'	
20	36'	Slightly compact gray silty fine and medium sand with lens of silt.
	39'	
450	42.5'	Very dense gray quartz sand all grades and fine gravel.
43	46'	Same material with larger gravel fine to medium.
	47'	Compact fine silty sand.

At 45' artesian water flow - 1 gal. per 1½ min.
Water table - Ground elev.

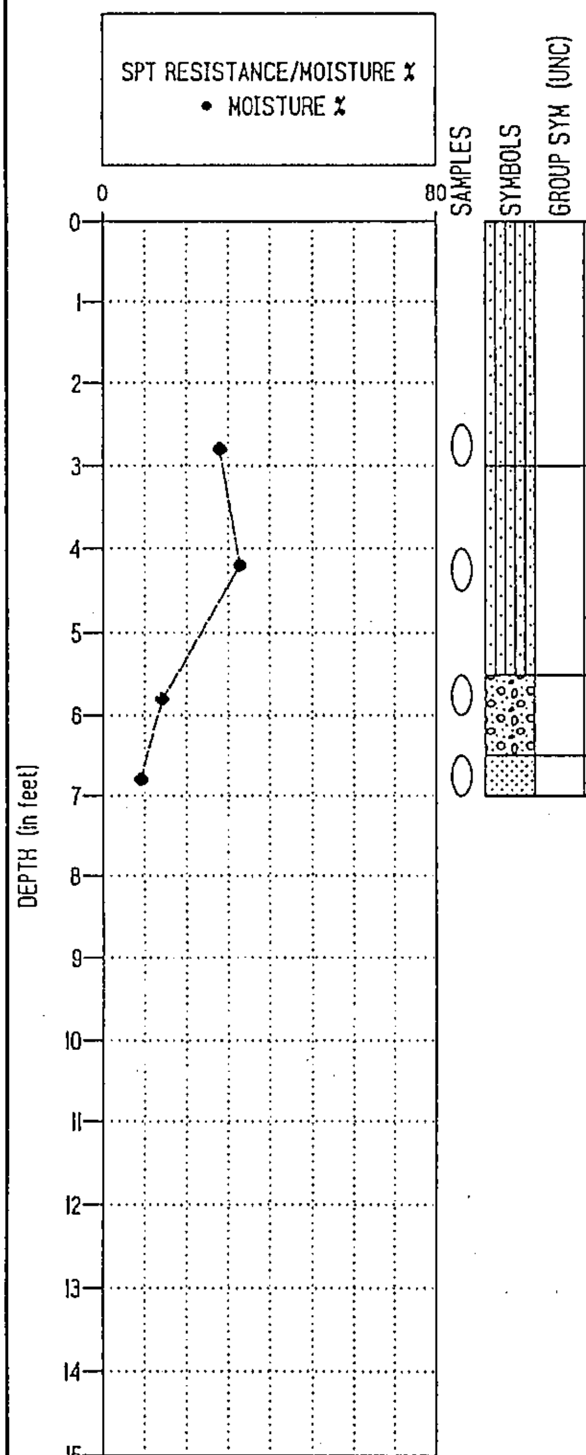
TEST HOLE H-2

Sta. 4+79 - 14' Rt ℄.

Sta. L 1054+75, 85' Lt

Fig. A-7

HONG WEST & ASSOCIATES
TEST PIT LOG



DESCRIPTION

loose to medium dense, brown silty fine to medium SAND with roots; weathered and interbedded with gray fine to med. sand lenses and seams (Outwash)

loose, gray, fine to medium, silty SAND; some roots. water bearing at 5.0 feet

dense, gray, well graded clean gravel; gravel rounded and waterbearing with some large cobbles

dense, gray, fine to medium sand and cobbles; sand is similar to sandstone

End of Test Pit

Sta. L 1053+55, 65' Lt

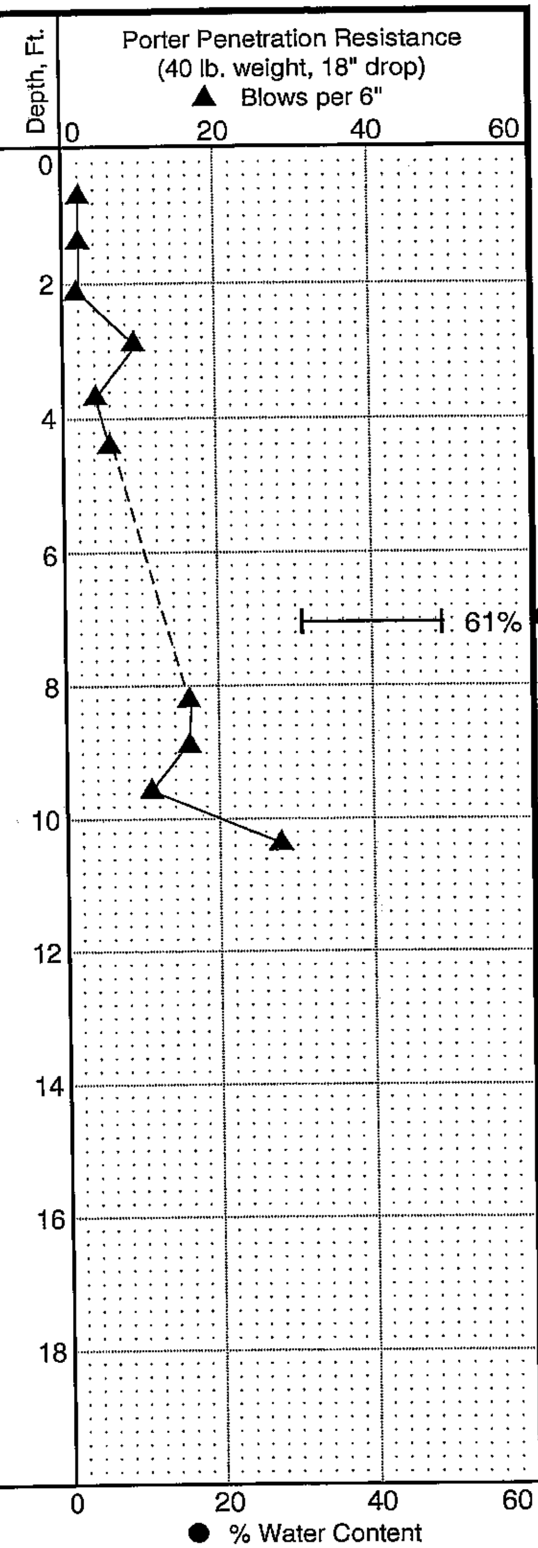
PROJECT NAME: SR-167: 15th Street SW to S. Grady Way
LOCATION: SR-167/S. 180th Interchange; AL2 ramp
PROJECT NUMBER: 91101
LOGGED BY: pls

TEST PIT TP-5

DATE DRILLED: 08/29/91
SURFACE ELEVATION: 26.5 ft.
TOTAL DEPTH: 6.5 ft.

Fig. A-8

SOIL DESCRIPTION Surface Elevation: Approx. 14.5 Feet	Depth, Ft.	Samples	Ground Water
Very loose, brown, trace to slightly gravelly, slightly clayey, fine to medium sandy SILT; roots to 1/2"; wood fragments; moist to wet	0	1	
Woody vegetation mat	1.5		
Medium stiff, mottled gray brown, clayey, fine sandy SILT; iron oxide staining; fine roots; moist to wet	2	2	▽
Soft to loose, brown to black, fine sandy, peaty SILT to peaty, fine SAND; charcoal, fine roots, and soft, brown, clayey to silty PEAT; wet	3.5	3, 4	
Medium stiff to stiff, gray, silty CLAY/clayey SILT; wet	6.5	5	
Medium dense, black to dark brown, trace gravel, clean to slightly silty, fine to medium SAND; thin dark brown silt layers from 10'-10.5'; wet	7.5	6, 7	
BOTTOM OF BORING COMPLETED 12-31-92	10.5		



LEGEND

- Porter split spoon sample
- Thin-wall tube sample
- * Sample not recovered
- Atterberg limits: Liquid limit, Natural water content, Plastic limit
- Impervious seal
- ▽ Water level
- Piezometer tip
- P Sample pushed

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

SR 167 H.O.V. Lanes
Renton, Washington

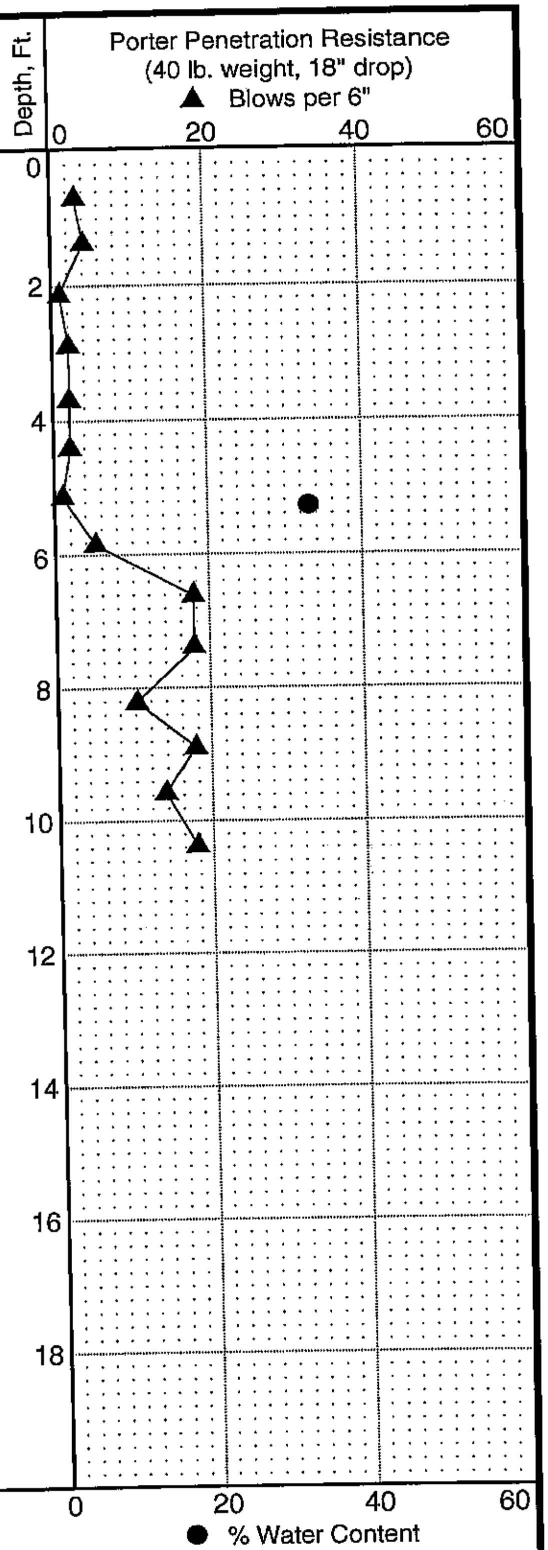
LOG OF HAND BORING HB 7-1
WALL NO. 7
STA 1082+05, 75' Rt. L

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-9

SOIL DESCRIPTION Surface Elevation: Approx. 16 Feet	Depth, Ft.	Samples	Ground Water
Soft, gray green to brown, trace fine gravel, slightly fine sandy, clayey SILT; iron oxide staining; roots to 1/4"; wood fragments; moist to wet	0 1.5	1	∇
Very loose, gray green, silty, gravelly SAND; wet	2	2	
Soft, brown, fine sandy, clayey SILT to silty CLAY; wood fragments with silty PEAT and loose sand layers; wet		3	
Soft, gray green, fine to medium sandy, silty CLAY; wood fragments	5	4	
Medium dense, black to dark brown, trace to slightly silty, fine SAND; thin dark brown SILT layers from 9.5'-10.5'	6	5 6 7	
BOTTOM OF BORING COMPLETED 12-31-92	10.5		



LEGEND

- Porter split spoon sample
- Thin-wall tube sample
- * Sample not recovered
- Impervious seal
- ∇ Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

Liquid limit
Natural water content
Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

SR 167 H.O.V. Lanes
Renton, Washington

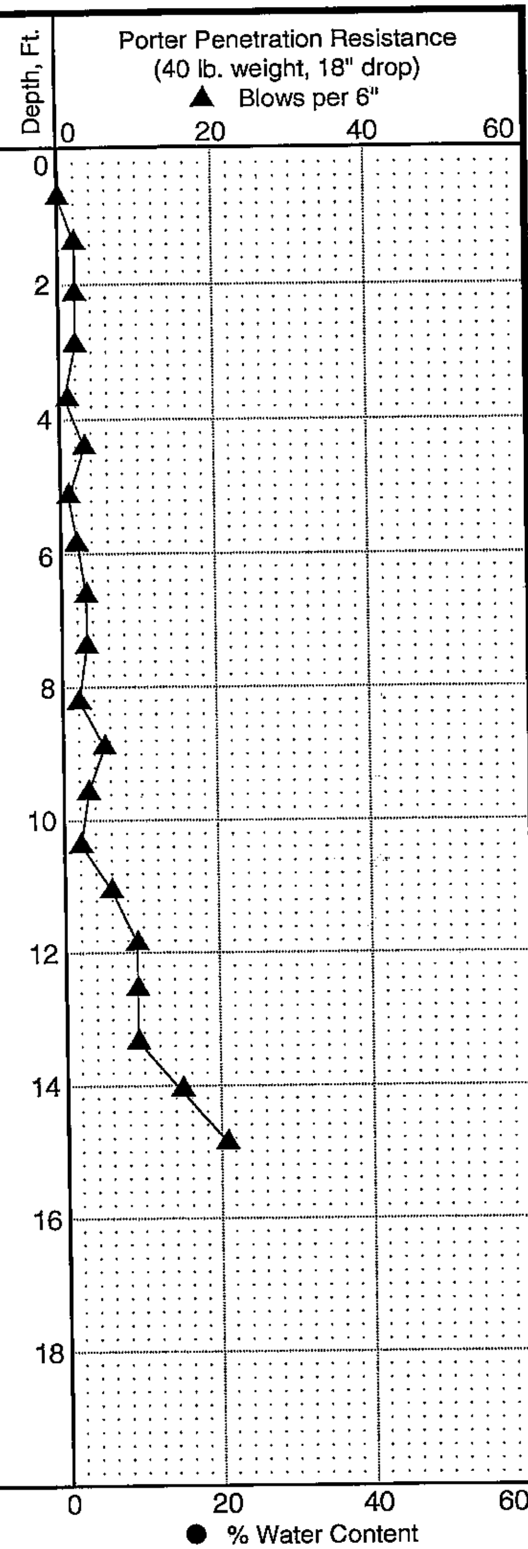
LOG OF HAND BORING HB 7-2
WALL NO. 7
STA 1079+00, 73' Rt. L

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-10

SOIL DESCRIPTION Surface Elevation: Approx. 20 Feet	Depth, Ft.	Samples	Ground Water
Very soft to soft, brown, fine sandy, clayey SILT; roots to 1/4"; wet	0	1, 2	▽
Very loose to soft, gray brown, trace fine gravel, silty, fine to medium SAND to fine sandy SILT	3	3	
Very loose, gray to gray brown, silty, fine to medium SAND; wood fragments with thin, soft, clayey SILT layers; wet	4.5	4, 5	
Very loose, gray green, trace fine gravel, slightly silty to silty, fine to medium SAND; trace wood fragments; wet	7.5	6, 7	
Medium stiff, fine sandy, SILT and CLAY; wet	10.5	8	
Loose to medium dense, silty, fine SAND; scattered wood fragments; wet	11.5	9, 10	
BOTTOM OF BORING COMPLETED 12-31-92	15		



LEGEND

- Porter split spoon sample
- Thin-wall tube sample
- * Sample not recovered
- Atterberg limits:
  - Liquid limit
  - Natural water content
  - Plastic limit
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

SR 167 H.O.V. Lanes
Renton, Washington

LOG OF HAND BORING HB 7-3
WALL NO. 7
STA 1076+10, 78' Rt. L

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-11

SOIL DESCRIPTION Surface Elevation: Approx. 19.5 Feet	Depth, Ft.	Samples	Ground Water	Depth, Ft.	% Water Content (0 20 40 60)
Very soft, mottled gray brown, trace gravel, slightly fine sandy, clayey SILT; roots to 1/2"; wet	0	1 Grab Sample	∇	0, 2	
Very loose, gray brown to gray green, trace gravelly, silty, fine to medium SAND; wood fragments	3	2 Grab Sample 3 Grab Sample 4 Grab Sample		4, 6	
Loose to medium dense, gray green, slightly silty, sandy GRAVEL; gravel is angular basalt; wet	8	5 Grab Sample		8, 10	
BOTTOM OF BORING COMPLETED 1-2-93 NOTE Hand boring was augered.	12			12, 14	

0 20 40 60

● % Water Content

LEGEND

- Ⅰ Porter split spoon sample
- Ⅱ Thin-wall tube sample
- * Sample not recovered
- Impervious seal
- ∇ Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

SR 167 H.O.V. Lanes
Renton, Washington

LOG OF HAND BORING HB 7-4
WALL NO. 7
STA 1076+10, 90' Rt. L

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-12

SOIL DESCRIPTION Surface Elevation: Approx. 16 Feet	Depth, Ft.	Samples	Ground Water	Depth, Ft.
	0			0
Soft, organic SILT to CLAY				
	2	1 Grab Sample		2
Loose, gray, silty, fine to medium SAND; fine roots; wet				
		2 Grab Sample		
	6			6
Medium dense, dark gray to dark brown, silty fine SAND; wet		3 Grab Sample		
				8
	9	4 Grab Sample		
BOTTOM OF BORING COMPLETED 1-2-93				10
				12
				14
				16
				18

0 20 40 60

● % Water Content

NOTES

1. Hand boring was augered.
2. Water level 4"-6" above ground.

LEGEND

- Porter split spoon sample
- Thin-wall tube sample
- * Sample not recovered
- Atterberg limits:
  - Liquid limit
  - Natural water content
  - Plastic limit
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

SR 167 H.O.V. Lanes
Renton, Washington

LOG OF HAND BORING HB 7-5
WALL NO. 7
STA 1079+00, 95' Rt. L

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-13

SOIL DESCRIPTION Surface Elevation: Approx. 14.5 Feet	Depth, Ft.	Samples	Ground Water
Soft, brown PEAT	0		
Very loose, mottled gray brown, trace gravel, clayey, silty, fine SAND; wet	1	1 Grab sample	
Soft to very loose, black, peaty SILT and fine SAND; charcoal; wet	2 – 2.5		
Soft, brown, silty PEAT; wet		2 Grab sample	
Soft to medium stiff, gray CLAY	5.5 – 6		
Medium dense, black to dark brown, slightly silty, fine SAND; wet	6.5		

BOTTOM OF BORING
COMPLETED 1-2-93

Depth, Ft.: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18

0 20 40 60

● % Water Content

NOTES

1. Hand boring was augered.
2. Water level at ground surface.

LEGEND

- Porter split spoon sample
- Thin-wall tube sample
- * Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

SR 167 H.O.V. Lanes
Renton, Washington

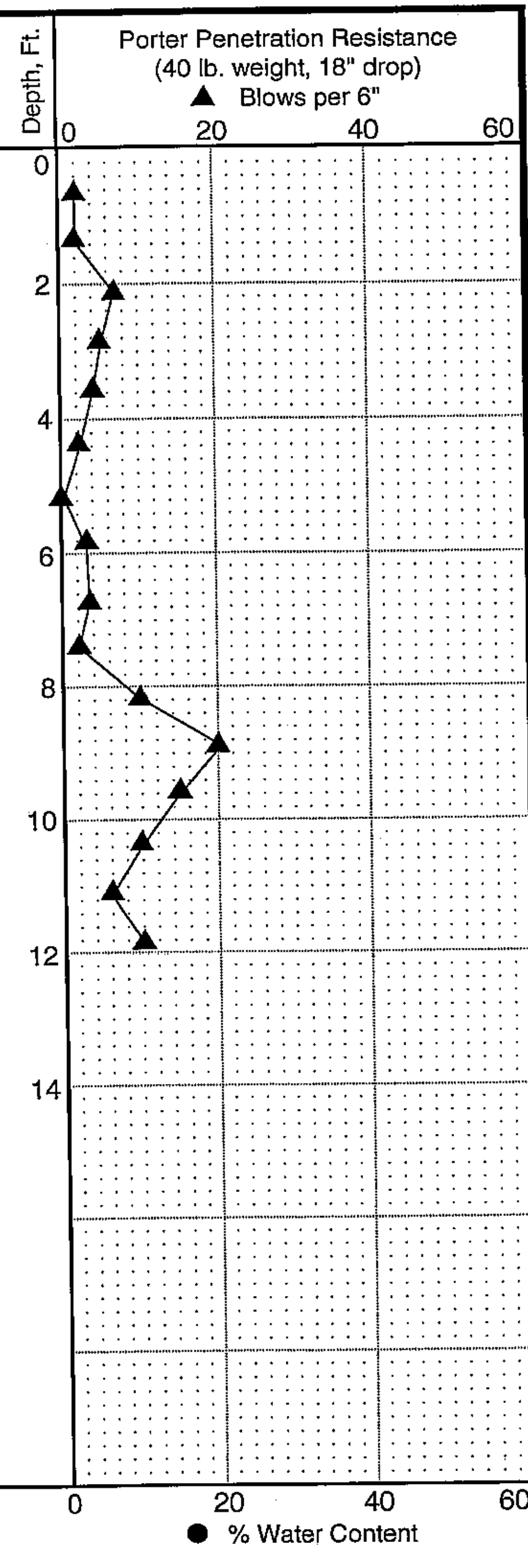
LOG OF HAND BORING HB 7-6
WALL NO. 7
STA 1082+05, 95' Rt. L

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-14

SOIL DESCRIPTION Surface Elevation: Approx. 24 Feet	Depth, Ft.	Samples	Ground Water
Soft, brown, fine sandy, clayey SILT; trace fine gravel; abundant fine roots; wood fragments; charcoal; moist	0	1	∇ During Drilling
Loose to very loose, gray to brown, trace gravel, silty, fine to medium SAND and loose, gray, silty, sandy GRAVEL; wood fragments; wet	1.5	2, 3, 4	
Soft to stiff, gray to brown, slightly fine sandy, clayey SILT; wet	7	5	
Loose to medium dense, gray, silty, fine to medium SAND; wood fragments; wet	8.5	6, 7, 8	
BOTTOM OF BORING COMPLETED 12-21-92	12		



NOTE

Could not auger past 6-inch diameter log at 10 feet.

LEGEND

- Porter split spoon sample
- Thin-wall tube sample
- * Sample not recovered
- Atterberg limits:
  - Liquid limit
  - Natural water content
  - Plastic limit
- Impervious seal
- ∇ Water level
- Piezometer tip
- P Sample pushed

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

SR 167 H.O.V. Lanes
Renton, Washington

LOG OF HAND BORING HB 7-7
WALL NO. 7
STA 1071+10, 85' Rt. L

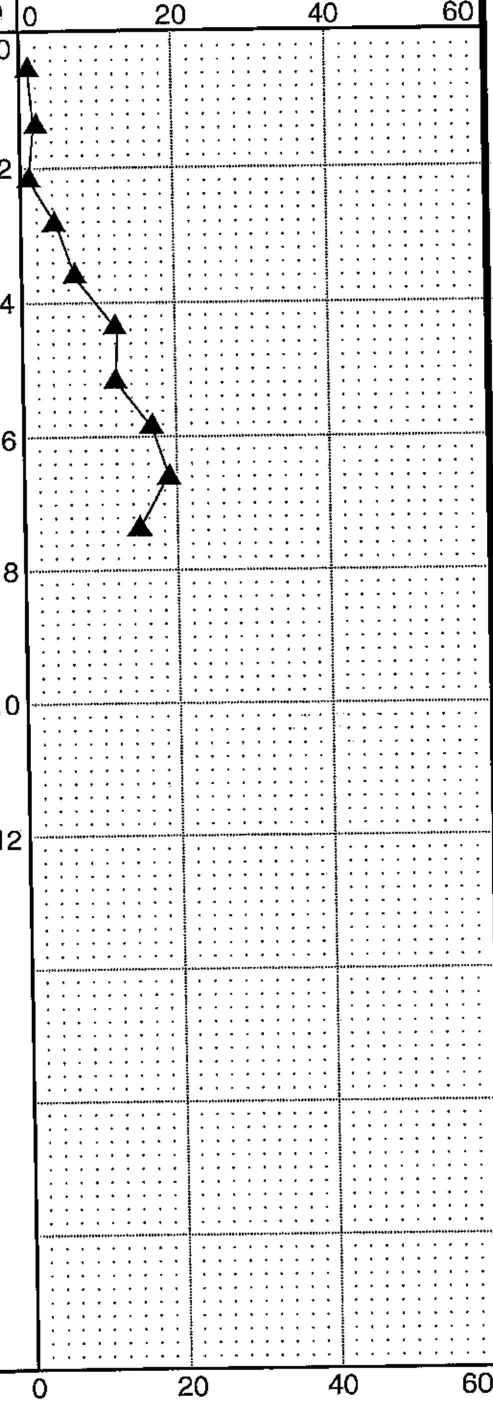
January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-15

SOIL DESCRIPTION Surface Elevation: Approx. 23 Feet	Depth, Ft.	Samples	Ground Water	Depth, Ft.
Very soft, brown to mottled gray, trace to slightly gravelly, slightly fine sandy, clayey SILT; fine roots; wood fragments; charcoal	0	1 2	During Drilling ▽	0 2
Medium to very stiff, mottled gray-brown to gray, slightly fine sandy, slightly clayey SILT to silty CLAY; iron-oxide stain at 3.0-4.0 feet; moist to wet with thin interbedded, medium dense, clayey, silty, fine SAND; wet	3.0	3 4 5		4 6
BOTTOM OF BORING COMPLETED 12-21-92 NOTE Clay too stiff to auger past 7.0 feet.	7.5			8 10 12

Porter Penetration Resistance
(40 lb. weight, 18" drop)
▲ Blows per 6"



● % Water Content

LEGEND

- Porter split spoon sample
- Thin-wall tube sample
- * Sample not recovered
- Impervious seal
- ▽ Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

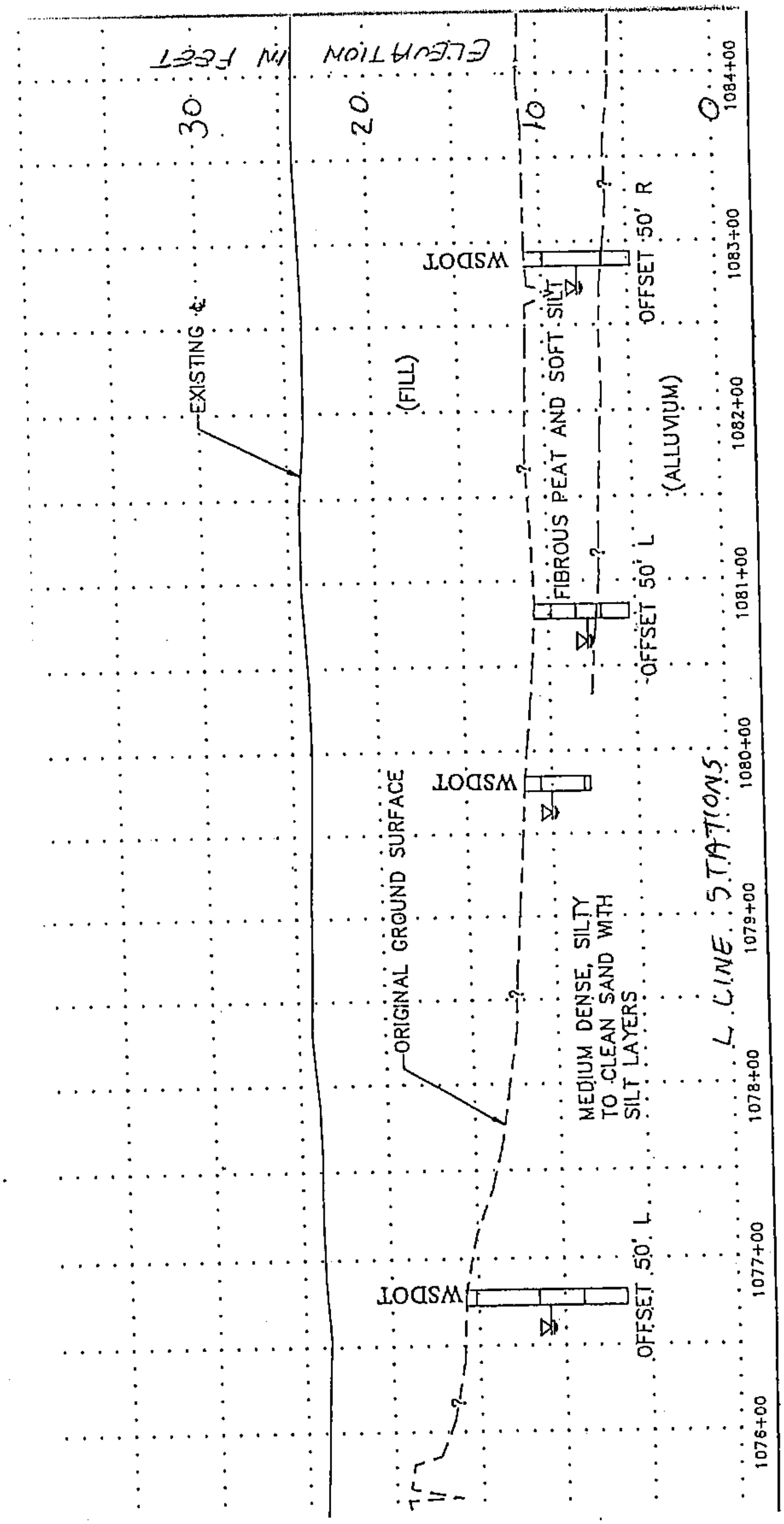
SR 167 H.O.V. Lanes
Renton, Washington

LOG OF HAND BORING HB 7-8
WALL NO. 7
STA 1068+75, 90' Rt. L

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-16

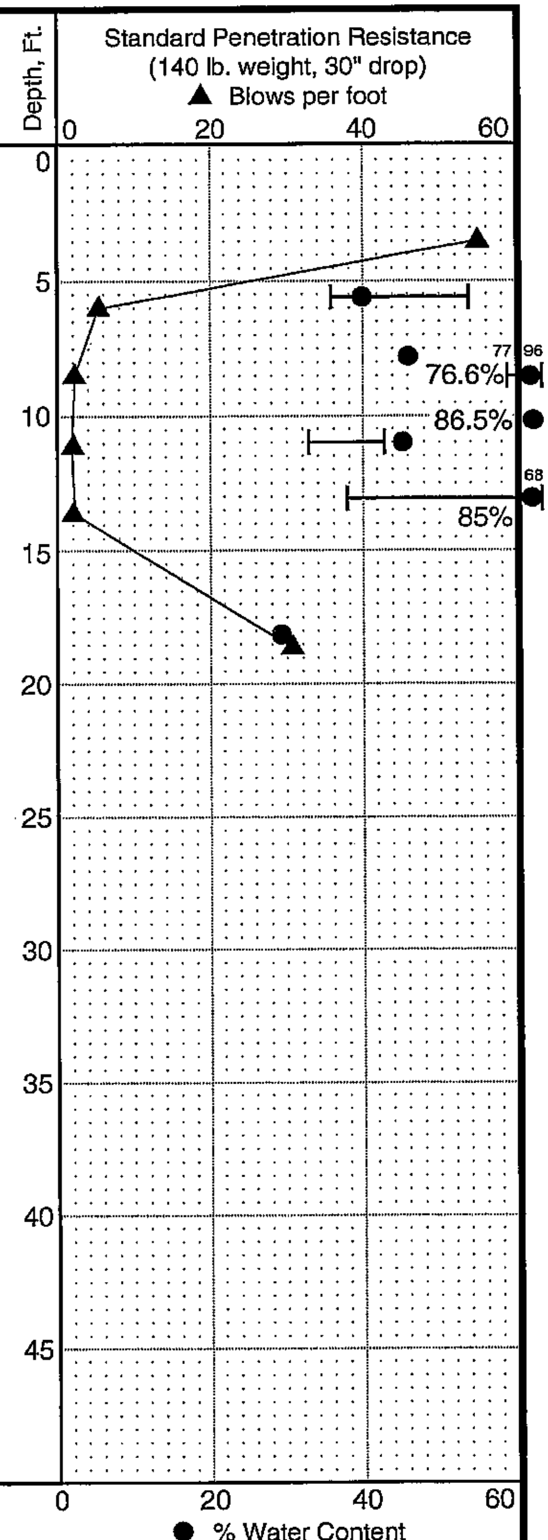


ELEVATION IN FEET
30
20
10
0
EXISTING ℄
(FILL)
WSDOT
FIBROUS PEAT AND SOFT SILT
(ALLUVIUM)
OFFSET 50' R
OFFSET 50' L
ORIGINAL GROUND SURFACE
MEDIUM DENSE, SILTY TO CLEAN SAND WITH SILT LAYERS
L LINE STATIONS
1076+00
1077+00
1078+00
1079+00
1080+00
1081+00
1082+00
1083+00
1084+00

Fig. A-17

SOIL DESCRIPTION Surface Elevation: Approx. 18.5 Feet	Depth, Ft.	Samples	Ground Water
Very dense, brown, silty, sandy GRAVEL; moist (FILL)	0	1	During Drilling ⊻
Medium stiff, green-gray, slightly clayey SILT to CLAY; moist	4.5	2	
Very loose, gray to brown, slightly silty, fine SAND; wet	6.3	3	
Very soft, gray to brown, organic, slightly clayey SILT to silty CLAY; scattered sandy zones; scattered to abundant wood fragments and large chunks of wood at 10'; moist	8.3	4, 5	
Dense, dark gray, trace to slightly silty, fine SAND; wet	17.5	6	
	19		

BOTTOM OF BORING
COMPLETED 12-11-92



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- * Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

SR 167 H.O.V. Lanes
Renton, Washington

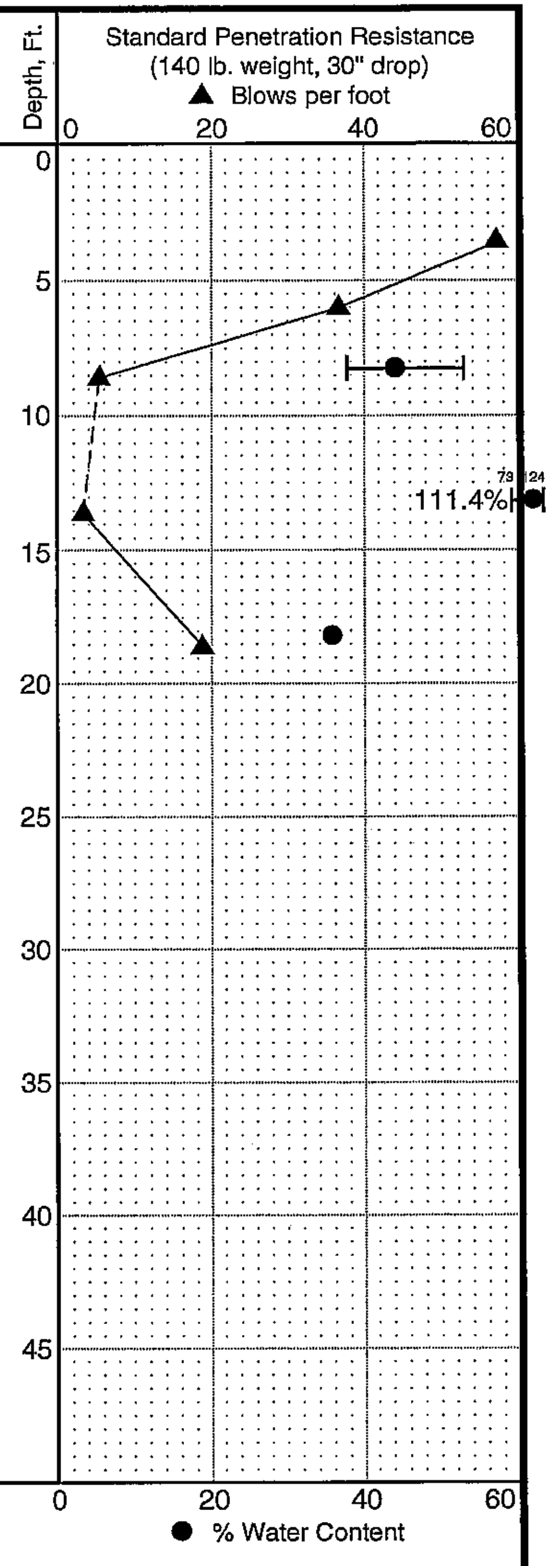
LOG OF BORING B 8-1
WALL NO. 8
STA. 990+00, 98' LT. EE

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-18

SOIL DESCRIPTION Surface Elevation: Approx. 17 Feet	Depth, Ft.	Samples	Ground Water
Dense to very dense, brown, silty, sandy GRAVEL; moist to wet (FILL)	0	1, 2	
Soft to medium stiff, gray to brown, organic, slightly clayey to clayey SILT; scattered to abundant plant and wood fragments; moist	6.5	3, 4, 5	During Drilling ▽
Medium dense to stiff, dark gray, silty, fine SAND and brown SILT; wet	14	6	
BOTTOM OF BORING COMPLETED 12-11-92	19		



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- * Sample not recovered
- Atterberg limits: Liquid limit, Natural water content, Plastic limit
- Impervious seal
- ▽ Water level
- Piezometer tip
- P Sample pushed

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

SR 167 H.O.V. Lanes
Renton, Washington

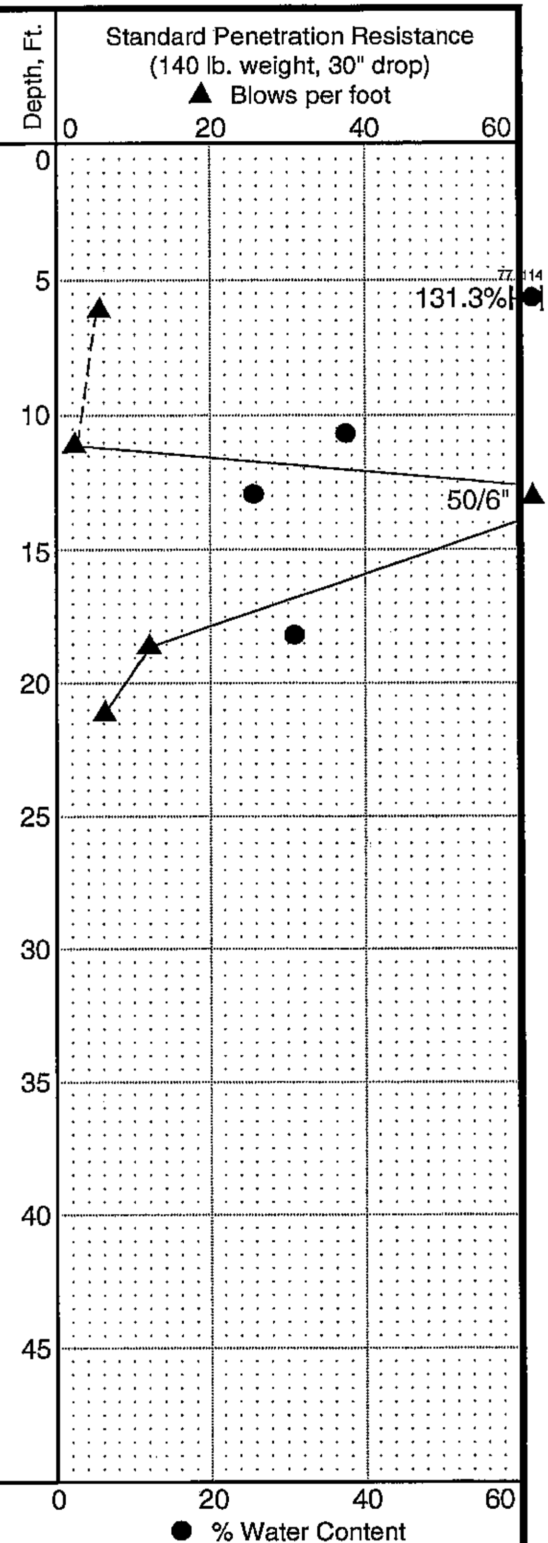
LOG OF BORING B 8-2
WALL NO. 8
STA. 987+00, 90' LT. EE

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-19

SOIL DESCRIPTION Surface Elevation: Approx. 14.5 Feet	Depth, Ft.	Samples	Ground Water
Medium dense to dense, brown, silty, sandy GRAVEL; gravel to 4"; moist to wet (FILL)	0		During Drilling ▽
Soft to medium stiff, brown, organic, slightly clayey SILT; abundant wood fragments	5	1	
Very loose to dense, dark gray, silty, fine SAND to fine sandy SILT; and very soft to stiff, gray-brown SILT to clayey SILT; wet	8.5	2, 3, 4, 5, 6	
BOTTOM OF BORING COMPLETED 12-11-92	21.5		



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- * Sample not recovered
- Impervious seal
- ▽ Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

SR 167 H.O.V. Lanes
Renton, Washington

LOG OF BORING B 8-3
WALL NO. 8
STA. 980+60, 96' LT. EE

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-20

LOG OF TEST BORING

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

S.H. ____ S.R. 167 SECTION S. 180th St to SR 405 Job No. [illegible]

Hole No. ____ Sub Section Sign Bridge No. 2 Cont. Sec. ____

Station EC 979+75 Offset 30.0' Rt of EOP Ground El. [illegible]

Type of Boring [illegible] Casing 3" x 19.0' W.T. El. 18.05' after pulling [illegible]

Inspector [illegible] Date 6-23-87/6-24-87 Sheet 1 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
.1			9	Sod
	P-1 31		14 / 17 Rec [illegible]	brown dense moist slightly gravelly, silty fine to medium SAND
			8	
	P-2 22		12 / 10 Rec 1.0'	brown medium dense moist slightly gravelly, silty fine to medium SAND
5				
	P-3 7		5 / 4 / 3 Rec 0.3	mottled wet loose, slightly silty fine to coarse SAND
			4	mottled wet medium dense, silty fine to medium SAND
	P-4 16		6 / 10 Rec 0.9'	w/traces of wood fragments
10				
	P-5 20		12 / 10 / 10 Rec 0.8	gray moist [illegible] sandy SILT
				-14.0' to -15.0' [illegible]
15				0.G. -15.0'
	P-6 5		4 / 2 / 3 Rec 0.8'	brown wet soft PEAT
			3	
	P-7 6		3 / 3 Rec 1.0'	gray wet soft clayey SILT w/ organic silt
15 20				

Sta. EE 979+80, 48' RT

DOT FORM 351-003 REVISED 12/79

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator
Copy to Fig. A-21A

H. F. 26.66-A (Rev. 5-67)

Sta EE 979+75 20.0' Rt of EOP

Hole No. Sub Section S 180th St to SR 405 Sheet 2 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
20			4	
	P-8		5 rec	gray wet medium dense slightly silty fine to medium
	10		5 1.0'	SAND w/silt lenses
			16	
	P-9		16 rec.	gray wet dense silty fine SAND
25	33		17 0.1'	
				Note: exist toe of slope is at approximate
				elevation 11.5'
				End of Boring at elevation -1.03'

Fig. A-21B

LOG OF TEST BORING

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

S.H. ______ S.R. 167 SECTION S. 180th St. to SR-405 NB HOV Lane Job No. L-8612

Cont. Sec. 1766

Hole No. PP-1 Sub Section ______

Ground El. 19.0'

Station SR-167 EE Line 981+00 Offset 25.0' Rt. of EOP

W.T. El. Not encountered

Type of Boring Portable Penetrometer Casing ______

Date January 21, 1987 Sheet 1 of 1

Inspector ______

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			4, 9, 23, 21 PORT PEN 1	SM, M.C.=16.1% Medium dense, brown, moist, gravelly, very silty, fine to medium SAND.
	16			
			7, 15, 25, 47 PORT PEN 2	SM, M.C.=15.2% Medium dense, brown, moist, gravelly, very silty, fine to coarse SAND.
	20			
5				
			26, 35, 36, 39 PORT PEN 3	No recovery. (Dense, brown, moist, gravelly, silty, fine to coarse SAND.)
	35			
				Note: blows per foot are equivalent to standard penetrometer values.
10				
				End of boring at 7.0' below ground elevation.
				This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.

Sta EE 981+00, 53' Rt

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator
Copy to Fig A-22

DOT FORM 351-003 REVISED 12/79

LOG OF TEST BORING

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

S.H. ______ S.R. 167 SECTION S. 180th St. to SR-405 - NB HOV Lane Job No. L-8612

Hole No. TP-2 Sub Section ______ Cont. Sec. 1766

Station EE Line 983+00 Offset 35.0' Rt. of EOP Ground El. 15.5'

Type of Boring Tripod - Wash Boring Casing 3" x 35.0' W.T. El. 11.7'

Inspector ______ Date January 22-23, 1987 Sheet 1 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	11		1, 4, 7, 10 — STD PEN 1	SM, M.C.=25.5% Medium dense, brown, wet, gravelly, very silty, fine to coarse SAND with organic material. Retained 1.0'
	7		3, 5, 2, 3 — STD PEN 2	SM, M.C.=18.6% Loose, brown, moist, gravelly, very silty, fine to coarse SAND. Retained 0.6'.
5		? ?	A, B — U-1	
	4		1, 2, 2, 3 — STD PEN 3	M.C.=199.0% Soft, brown, saturated, highly organic SILT with wood fragments. (Visual I.D. only) Retained 1.0'.
10			A, B, C, D, E — U-2	Brown, PEAT
	3		0, 1, 2, 1 — STD PEN 4	ML, M.C.=41.9% Very loose, brown, wet, fine sandy SILT. Retained 1.3'.
15			A, B, C, D	
	6		1, 4, 2, 1 — STD PEN 5	SP, M.C.=27.1% Loose, black, brown, moist, slightly silty, fine to medium SAND with fibrous material in tip of sampler. Retained 1.5'.
20				

Sta. EE 983+00, 65' Rt

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator

DOT FORM 351-003 REVISED 12/79

Copy to Fig A-23A

Hole No. TP 2 Sub Section S. 180th St. to SR-405 - NB HOV Lane Sheet 2 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	2		2 1 1 1 STD PEN 6	Very loose, gray, wet, fine sandy SILT. Retained 2.0'.
			A B C U-4	
25	32		14 17 15 STD PEN 7	GP Dense, gray, wet, slightly silty, fine to coarse sandy GRAVEL. Retained 0.7'.
30	23		8 11 12 STD PEN 8	GW Medium dense, gray, wet, slightly silty, fine to coarse sandy GRAVEL. REtained 0.4'.
35	26		9 12 14 STD PEN 9	GW/GM Dense, gray, wet, silty, fine to coarse sandy GRAVEL. Retained 0.3'.
40				
				End of boring at 36.5' below ground elevation.
				This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.

Fig A-23 B

LOG OF TEST BORING

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

S.H. ______ S.R. 167 SECTION S. 180th St. to SR-405 - NV HOV Lane Job No. L-8612

Cont. Sec. 1766

Hole No. TP-3 Sub Section ______

Station EE Line 986+00 Offset 50.0' Rt. of EOP Ground El. 11.8'

Type of Boring Tripod - Wash Boring Casing 3" x 30.0' W.T. El. 11.3'

Date Janaury 26-28, 1987 Sheet 1 of 2

Inspector ______

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	4		1, 1, 3, 5 — STD PEN 1	SILT (Duff) SM, M.C.=58.9% Loose, brown, wet, very silty, fine to medium SAND with organics. Retained 0.9'.
			A, B, C, D — U-1	Mottled, wet, slightly sandy, slightly clayey SILT.
5				
	2		1/12", 2, 1 — STD PEN 2	M.C.=417.3% Soft, brown, saturated, highly organic SILT. (Visual I.D. only) Retained 1.0'.
			A, B, C, D — U-2	A: OL, M.C.=60.9% Gray, wet, organic SILT.
10				
	19		5, 8, 11, 10 — STD PEN 3	SP/SM, M.C.=23.5% Medium dense, black, moist, silty, fine to medium SAND. Retained 2.0'.
	17		4, 8, 9, 3 — STD PEN 4	Medium dense, black, moist, silty, fine to medium SAND. Retained 1.2'.
15				
	2		1, 1, 1, 0 — STD PEN 5	ML, M.C.=38.9% Very loose, gray, wet, fine sandy SILT. Retained 1.0'.
			A, B, C, D, E, F — U-3	
20				

Sta. EE 986+00, 80' Rt.

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator
Copy to Fig A-24 A

DOT FORM 351-003 REVISED 12/79

Hole No. TP-3 Sub Section S. 180th St. To SR-405 - NB HOV Lane Sheet 2 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	27		16 16 11 19 STD PEN 6	GW Dense, gray, wet, slightly silty, fine to coarse sandy GRAVEL. Retained 2.0'.
25	35		23 20 15 16 STD PEN 7	Dense, gray, wet, slightly silty, fine to coarse sandy GRAVEL. Retained 0.2'.
30	28		20 13 15 22 STD PEN 8	Dense, gray, wet, slightly silty, fine to coarse sandy GRAVEL. Retained 0.7'.
35				End of boring at 32.0' below ground elevation.
				This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.

DOT FORM 351-003A REVISED 4/80

Fig A-24B

LOG OF TEST BORING

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

S.H. ______ S.R. 167 SECTION S. 180th St. to SR-405 - NB HOV Lane Job No. L-8612

Hole No. PP-4 Sub Section ______ Cont. Sec. 1766

Station EE Line 988+00 Offset 57.0' Rt. of EOP Ground El. 12.5'

Type of Boring Portable Penetrometer Casing ______ W.T. El. 11.5'

Inspector ______ Date February 13, 1987 Sheet 1 of 1

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	8		3 8 9 6 PORT PEN 1	SM, M.C.=31.7% Loose, brown, wet, silty, fine to coarse SAND.
	1		2 2 2 3 PORT PEN 2	ML, M.C.=73.3% Very loose, brown-gray, wet, saturated, fine to medium sandy SILT.
5			A B C U-1	A: OL, M.C.=64.9% Gray, wet, organic SILT.
	1		3 3 2 4 PORT PEN 3	M.C.=53.9% Very soft, brown, wet, very silty ORGANICS. (Visual I.D. only).
10			A B C D E F U-2	Gray, wet, clayey SILT.
	16		10 15 18 16 PORT PEN 4	Medium dense, black, brown, moist, silty, fine SAND. (Insufficient material - Visual I.D. only)
15	8		5 9 7 8 PORT PEN 5	No recovery.
				Note: blows per foot are equivalent to standard penetrometer values. End of boring at 16.5' below ground elevation.
20				This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.

Sta EE 988+00, 87' Rt

DOT FORM 351-003 REVISED 12/79

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator
Copy to Fig. A-25

LOG OF TEST BORING WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

S.H. ______ S.R. 167 SECTION S. 180th St. to SR-405 - NB HOV Lane Job No. L-8612

Hole No. TP 5 Sub Section ______ Cont. Sec. 1766

Station EE Line 990+30 Offset 58.0" Rt. of EOP Ground El. 14.0'

Type of Boring Tripod - Wash Boring Casing 3" x 30.0' W.T. El. 11.2'

Inspector ______ Date January 30-February 1, 1987 Sheet 1 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.		DESCRIPTION OF MATERIAL
	1		1 1/ 12" 1	STD PEN 1	SM, M.C.=35.0% Very loose, brown wet, very silty, fine to medium SAND with organic material. Retained 0.6'.
			A B C D	U- 1	Mottled, moist, slightly sandy SILT with wet sand lenses.
5					approximate flow line of ditch
	4		1 1 3 4	STD PEN 2	M.C.=37.3% Soft, brown, wet, silty ORGANICS. (Visual I.D. only) Retained 1.5'. Ph and resistivity taken
			A B C	U- 2	A: OL, M.C.=90.3% Gray, saturated, organic SILT B: OL, M.C.=85.9% Gray, saturated, organic SILT
10					
	4		2 2 2 6	STD PEN 3	ML, M.C.=41.1% Very loose, gray, wet, fine sandy SILT. Retained 1.5'.
			A	U- 3	
15	10		8 6 4 2	STD PEN 4	No recovery. (Medium dense, gray, wet, slightly silty, fine to medium SAND.
	3		1 1 2 2	STD PEN 5	ML, M.C.=41.4% Very loose, gray, wet, fine to medium sandy SILT. Retained 0.8'.
20					

Sta. EE 990+30, 87' Rt.

DOT FORM 351-003 REVISED 12/79

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator
Copy to Fig. A-26 A

Hole No. TP 5 Sub Section S. 180th St. to SR-405 - NB HOV Lane Sheet 2 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	18		9 9 9 9 STD PEN 6	GM Medium dense, gray-brown, wet, fine to coarse sandy, very silty GRAVEL. Retained 0.4'.
25	11		10 3 8 17 STD PEN 7	GW/GM Medium dense, gray-brown, wet, silty, fine to coarse sandy GRAVEL. Retained 0.5'.
30	34		20 15 19 20 STD PEN 8	No recovery.
35				End of boring at 32.0' below ground elevation.
				This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.

Fig. A-26 B

DOT FORM 351-003A REVISED 4/80

LOG OF TEST BORING

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

S.H. ______ S.R. 167 SECTION S. 180th St. to SR-405 - NB HOV Lane Job No. L-8612

Hole No. TP 6 Sub Section ______ Cont. Sec. 1766

Station DR 2 81+80 Offset 32.0' Rt. New ℄ Ground El. 16.0'

Type of Boring Tripod - Wash Boring Casing 3" x 27.5' W.T. El. 13.2'

Inspector ______ Date February 9-10, 1987 Sheet 1 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	2		1 1 1 2 STD PEN 1	SM, M.C.=24.4% Very loose, brown, moist, very silty, fine to medium SAND. Retained 0.6'.
			A B C D E U-1	D: OL, M.C.=46.4% Gray, wet, organic SILT. E: OL, M.C.=45.6% Gray, wet, organic SILT.
5	1		1/12" 1 2 STD PEN 2	OL, M.C.=133.0% Very loose, gray-brown, saturated, highly organic, fine to medium sandy SILT. Retained 0.6".
			A B C U-2	
10	8		2 4 4 4 STD PEN 3	SM, M.C.=27.9% Loose, gray-brown, moist, silty, fine SAND. Retained 2.0'.
			A B C D E U-3	
15	7		7 6 1 3 STD PEN 4	SP/SM, M.C.=21.8% Loose, black, brown, moist, silty, fine to coarse SAND. Retained 0.5'.
	2		1 1 1 1 STD PEN 5	SM, M.C.=32.3% Loose, gray, wet, very silty, fine SAND. Retained 1.0'.
20				

Sta EE 992+25, 105' Rt

DOT FORM 351-003 REVISED 12/79

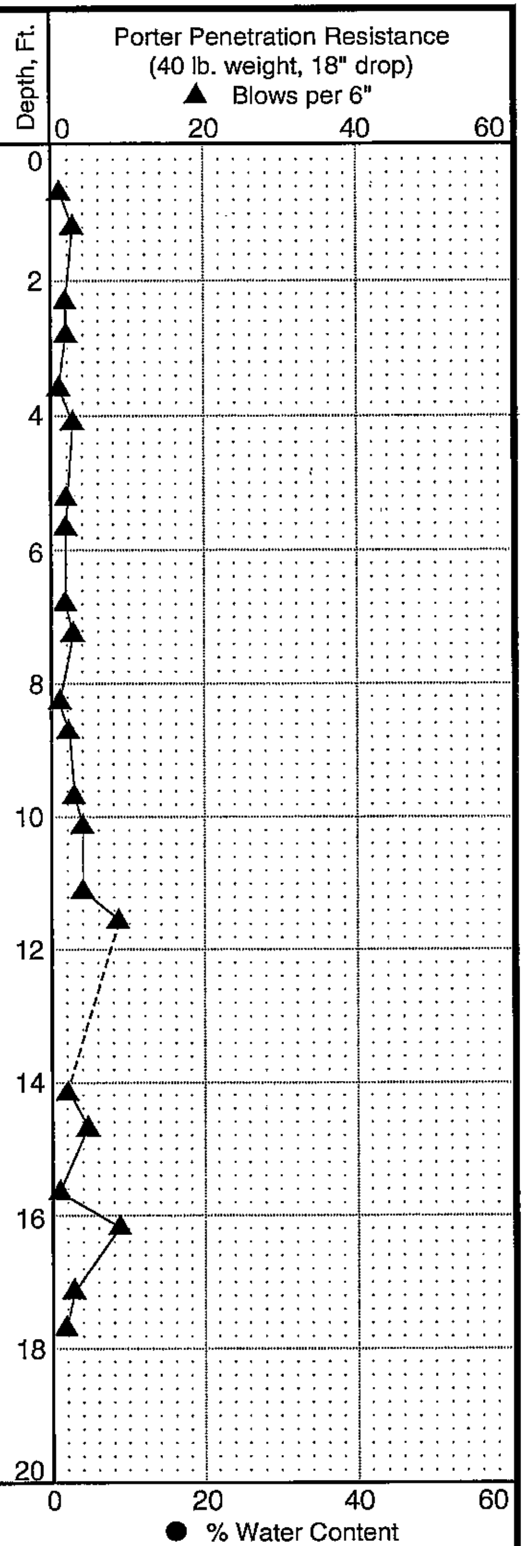
Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator
Copy to Fig. A-27 A

Hole No. TP 6 Sub Section S. 180th St. to SR-405 - NB HOV Lane Sheet 2 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			A B U-4	
			15 10 9 9 STD PEN 6	GP/GM Medium dense, gray, wet, slightly silty, fine to coarse sandy GRAVEL. Retained 0.7'.
	19			
25				
			15 17 17 STD PEN 7	Dense, gray, wet, slightly silty, fine to coarse sandy GRAVEL. Retained 0.7'.
	34			
30				
				End of boring at 29.0' below ground elevation.
				This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.

Fig. A-27 B

SOIL DESCRIPTION Surface Elevation: Approx. 17 Feet	Depth, Ft.	Samples	Ground Water
	0	1	During Drilling ∇
Soft to very soft, brown to mottled gray-brown, slightly clayey, fine sandy SILT to SILT; fine roots; wood fragments; iron-oxide stain; moist to wet		2, 3, 4, 5	
	7		
Loose to very loose, gray, silty fine SAND; wet		6	
	8.5		
Soft, gray, slightly clayey, fine sandy SILT; wood fragments; wet		7	
	11	8	
Medium dense to very loose, gray, slightly silty to silty, fine to medium SAND; marsh odor; wet		9 P	
	13.5		
Soft to medium stiff, gray-brown, slightly fine sandy, clayey SILT; wood fragments; wet		10	
	15	11	
Loose, black, interbedded, slightly silty, fine to medium SAND; wood fragments; wet; scattered soft, brown PEAT; wet; soft, gray-brown, slightly clayey, slightly fine sandy SILT; wood fragments; wet		12, 13	
BOTTOM OF BORING COMPLETED 12-18-92	20	14	



LEGEND

- Porter split spoon sample
- Thin-wall tube sample
- * Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:
- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

SR 167 H.O.V. Lanes
Renton, Washington

LOG OF HAND BORING HB 9-1
WALL NO. 9
STA 79+25, 35' RT AR2

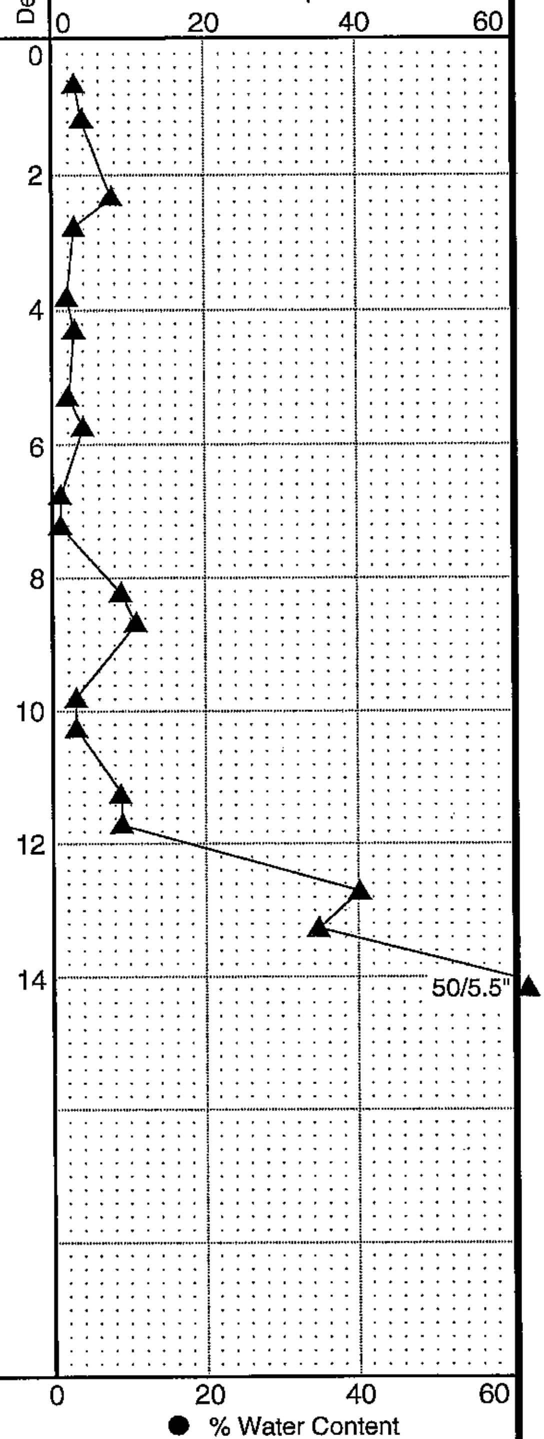
January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-28

SOIL DESCRIPTION Surface Elevation: Approx. 14 Feet	Depth, Ft.	Samples	Ground Water
Soft to medium stiff, brown to mottled gray-brown, fine sandy SILT; iron-oxide stain; scattered fine roots; wet	0	1, 2, 3	During Drilling ∇
Loose to very loose, gray, slightly clayey, silty, fine SAND; wet	4.5	4, 5	
Loose to medium dense, gray, silty fine to medium SAND; coarse sand and fine gravel at 11.0 to 12.0 feet; scattered wood fragments; wet	7.5	6, 7, 8	
Dense to very dense, gray, slightly silty, sandy fine GRAVEL; wet	12	9, 10	
BOTTOM OF BORING COMPLETED 12-21-92	14.5		

Porter Penetration Resistance
(40 lb. weight, 18" drop)
▲ Blows per 6"



LEGEND

- Porter split spoon sample
- Thin-wall tube sample
- * Sample not recovered
- Atterberg limits: Liquid limit, Natural water content, Plastic limit
- Impervious seal
- ∇ Water level
- Piezometer tip
- P Sample pushed

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

SR 167 H.O.V. Lanes
Renton, Washington

LOG OF HAND BORING HB 9-1
WALL NO. 9
STA 81+44, 57' RT AR2

January 1993 W-6391-03

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

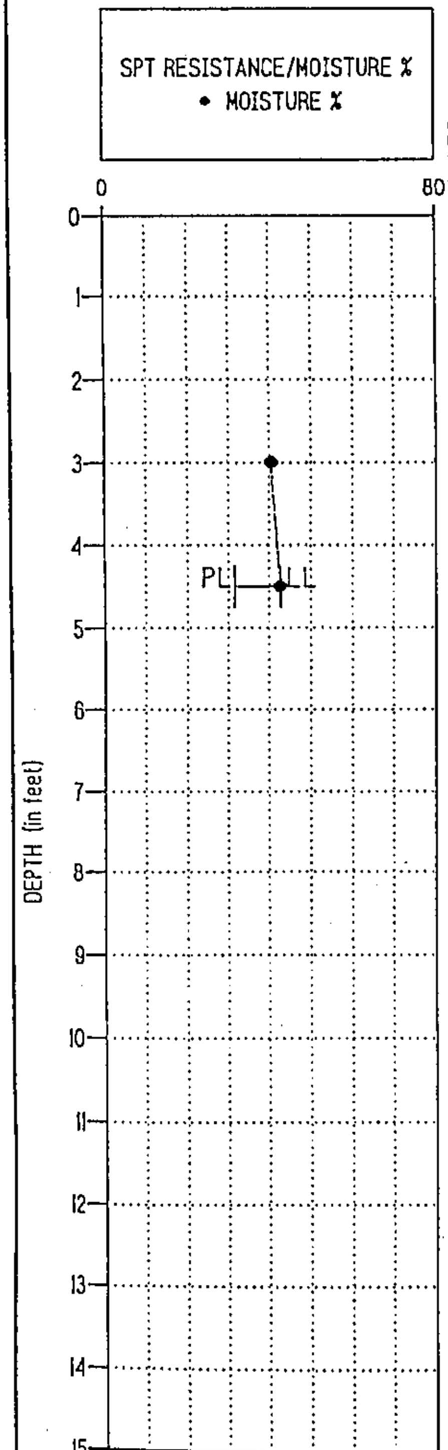
FIG. A-29

TP-7 Sta. AR2 78+10, 18' Rt.

0.0 to 1.5 ft	Loose, dark brown to brownish-gray fine sandy SILT, some roots and organics
1.5 to 4.0 ft	Loose, brown to reddish-brown, silty fine SAND; to fine to medium SAND, some silt
4.0 to 7.0 ft	Very soft, gray to greenish-gray, SILT
7.0 to 9.0 ft	Compact, gray, fine to medium SAND, trace to little silt
9.0 to 15.0	Very soft to soft, gray to greenish-gray, SILT
	Test pit was terminated at 15.0 ft depth. Slight to moderate groundwater seepage observed at a depth of 6.0 ft (about elev. 10.0) on 31 March 1988. Sand layer between the silts sloughed readily (within 10 minutes) once the groundwater table was encountered.

Fig. A-30

HONG WEST & ASSOCIATES
TEST PIT LOG



SAMPLES	SYMBOLS	GROUP SYM (UNC)	DESCRIPTION	PIEZOMETER
O			Loose, brown to gray with reddish mottling, silty SAND. Some roots to a depth of 3 feet. (FILL)	
O		ML	Very soft, blue gray, SILT. Plastic. Moist to wet. Increasing sand fraction with depth.	
O			Loose, gray, SAND. Trace to gravelly. Wet. Water table at 8.5 feet.	▼

End of pit at 9 feet.

Test pit sides caved in below 6 feet, progresses upward.

Sta. ARZ 83+40, 47' Rt.

PROJECT NAME: SR-167: 15th Street SW to S. Grady Way
LOCATION: SR-167/SR-405 Interchange
PROJECT NUMBER: 91101
LOGGED BY: pls

TEST PIT TP-1

DATE DRILLED: 08/28/91
SURFACE ELEVATION: 15 ft.
TOTAL DEPTH: 9 ft.

Fig. A-31

APPENDIX B

LABORATORY TESTING

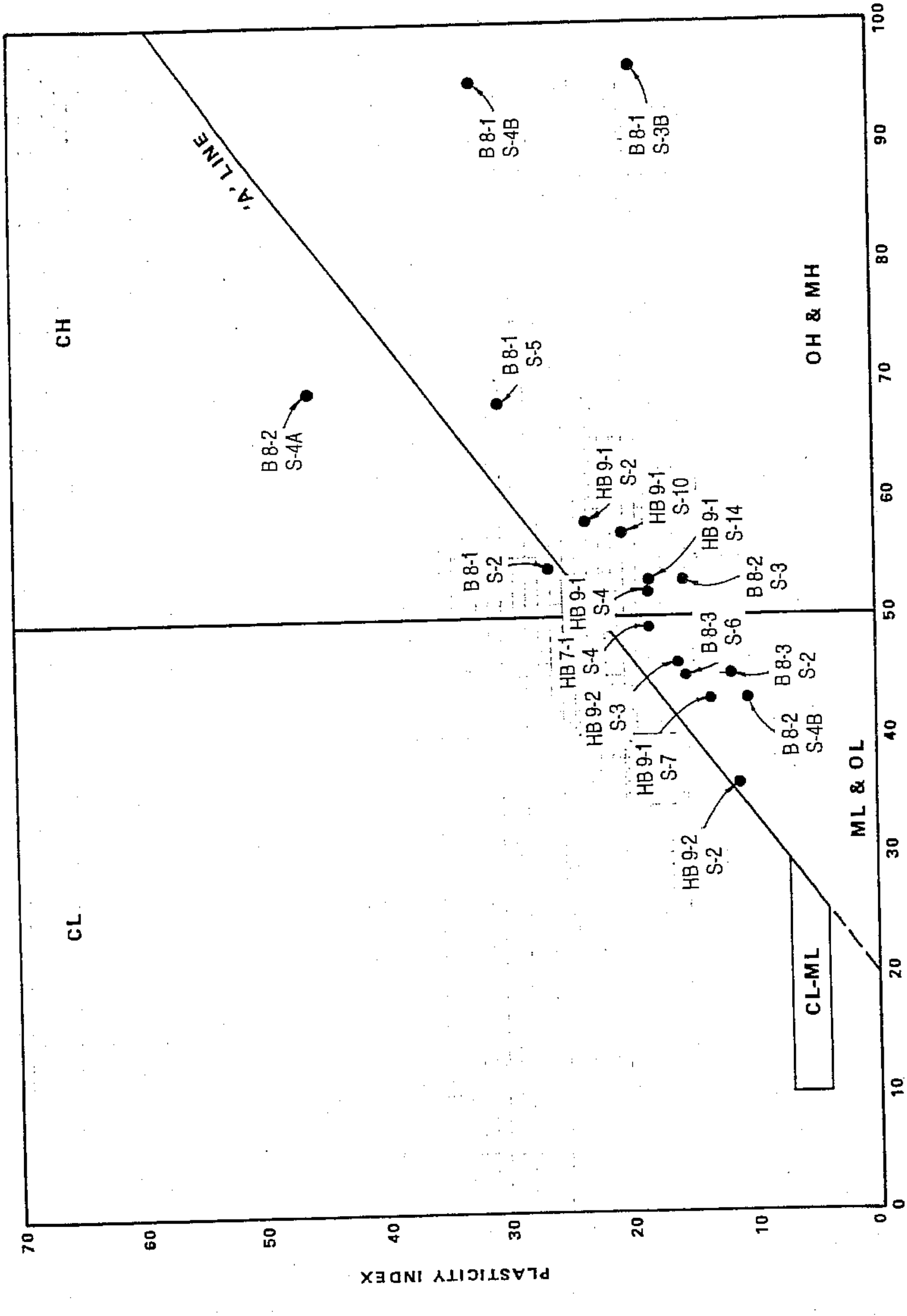
APPENDIX B

LABORATORY TESTING

Samples retrieved from explorations conducted for this study were returned to Shannon & Wilson's laboratory for more detailed visual examination. In addition, a number of these samples were selected for index and engineering properties testing. The index properties are used to correlated the site soils with materials from other locations which have been tested in greater detail for engineering properties. The index and engineering properties testing were performed by Hong West & Associates, Inc., of Lynnwood, Washington.

Index testing of the site soils included water-content and Atterberg limit determinations. The results of the water-content and Atterberg limit determinations are presented on the logs of the individual borings in Appendix A. In addition, the Atterberg limits are presented on Figure B-1. Atterberg-limit testing was conducted on selected samples to provide information on the plasticity characteristics of the fine-grain soils encountered in the explorations.

Determination of engineering properties consisted of two consolidation tests. Tests were performed on peat samples taken from the vicinity of wall 8. These tests were conducted to evaluate the potential settlement of the embankment fill and proposed retaining wall. The results from these tests are presented in Figures B-2 to B-7. A description of the shelby-tube sample from which they were retrieved are shown on Figures B-8 and B-9.



PLASTICITY CHART

BORING	SAMPLE	LL	PL	PI
B 8-2	S-5	124	73	51
B 8-3	S-1	114	77	37

SR167 H.O.V. Lanes
Renton, Washington
PLASTICITY CHART

January 1993

W-6391-03

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. B-1

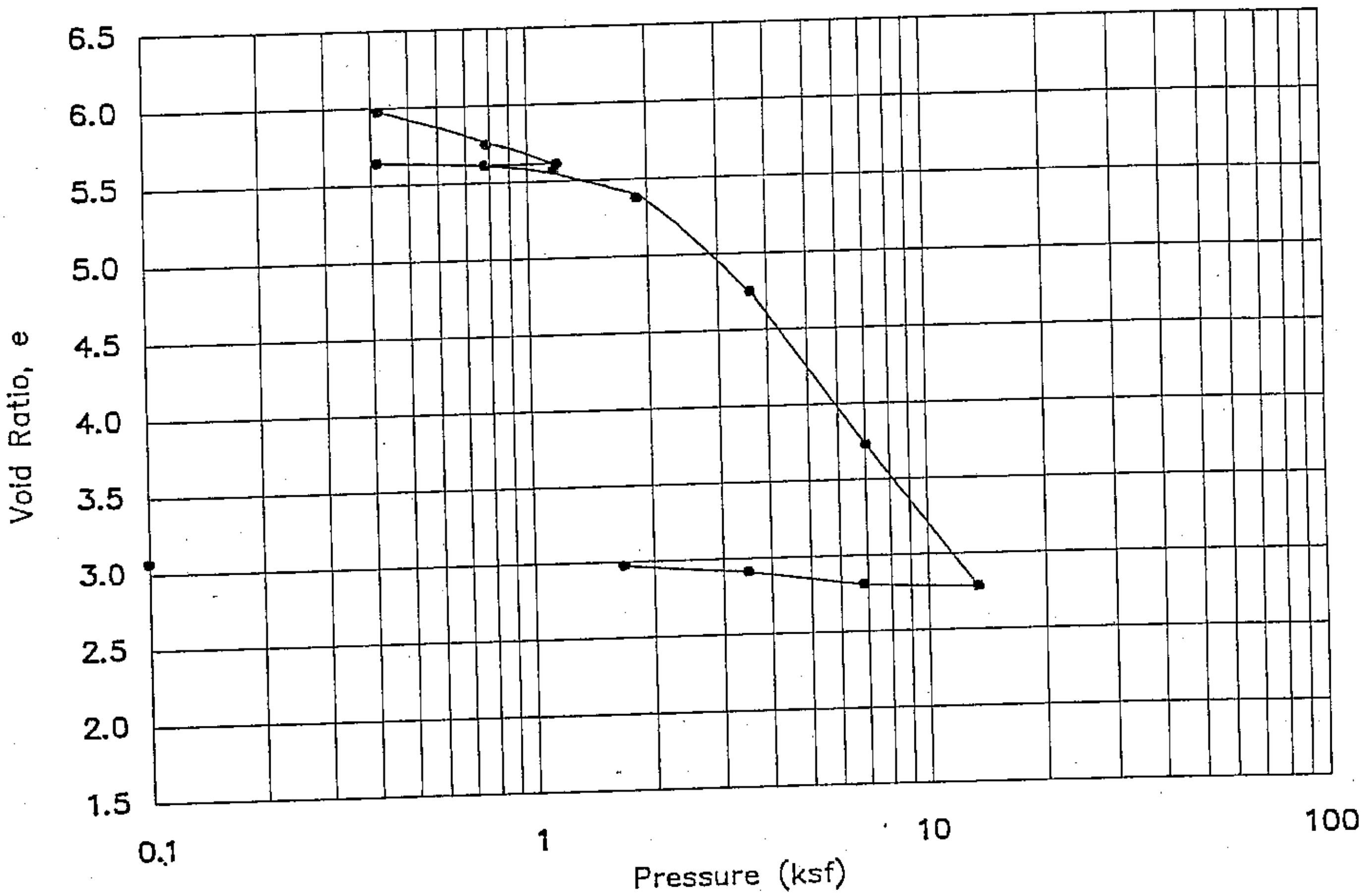
HONG WEST & ASSOCIATES, INC.

CONSOLIDATION TEST RESULTS

Project: SR 167
Location: King County, Washington

Project Number: 92050-37
Date Tested: 12-18-92

Test No. 1
Boring: 8-2
Depth (ft.): 7.5 - 9.0'
Diameter (in.): 2.416
Assumed Sp. Gravity: 1.9

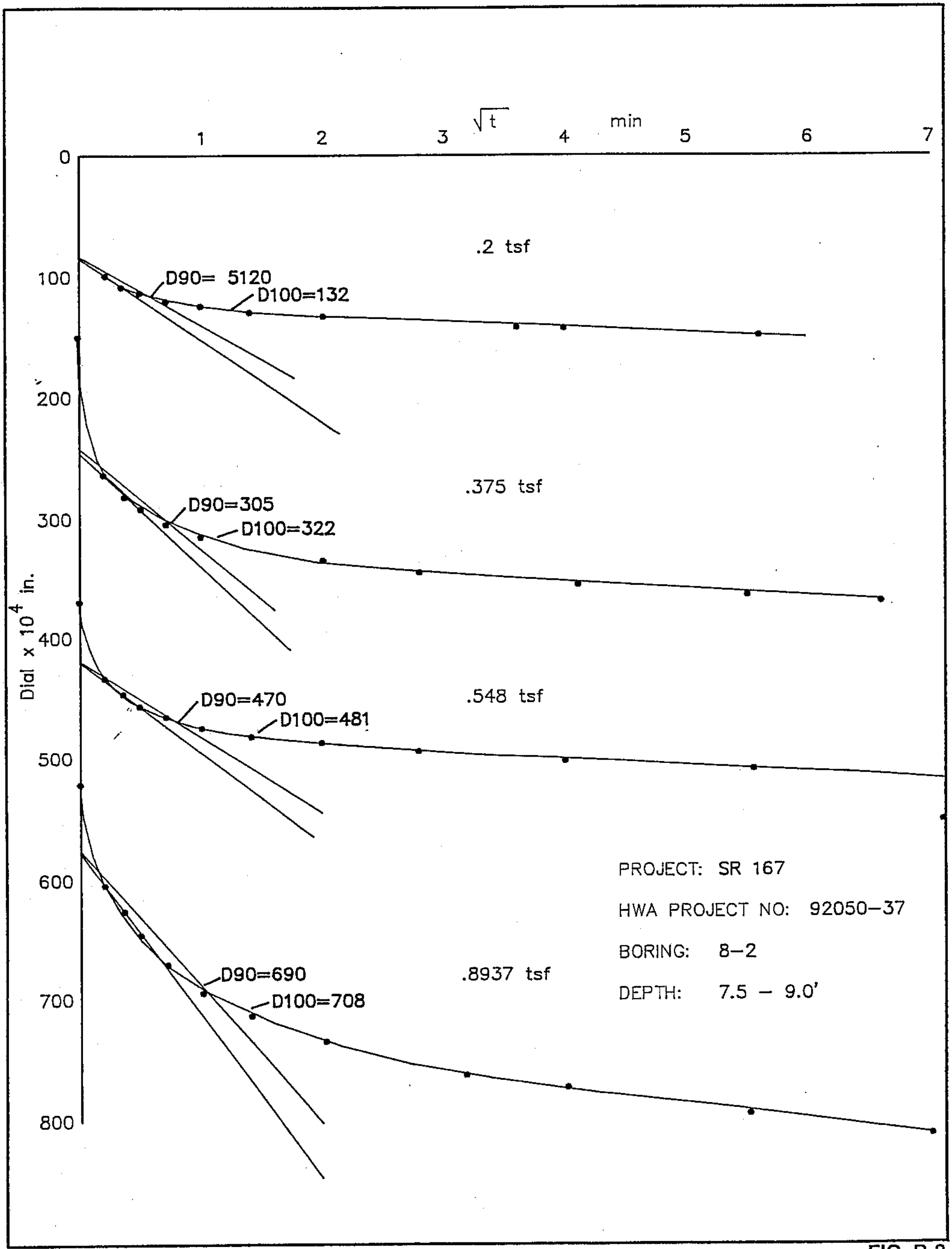


	INITIAL	FINAL
Height (in.)	1.0	0.5769
Water Content %	299%	145%
Wet Density (pcf)	66.6	72.4
Dry Density (pcf)	16.7	29.5
Saturation %	91.4	100%

Sample Description: Soft, dark brown PEAT (PT)

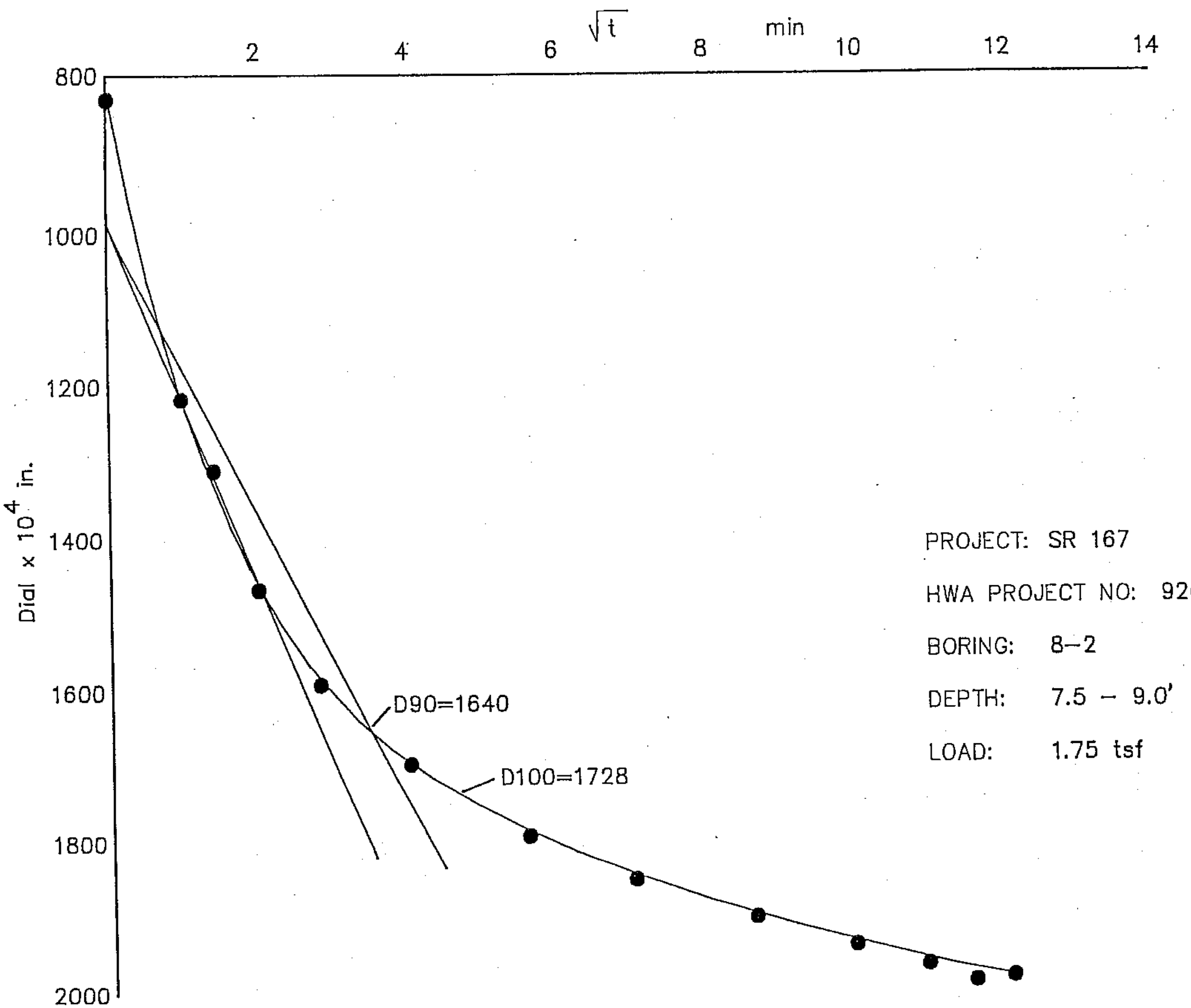
Liquid Limit: N/A
Plastic Limit: N/A

FIG. B-2



PTY-A-004.0

FIG. B-3



PROJECT: SR 167

HWA PROJECT NO: 92050-37

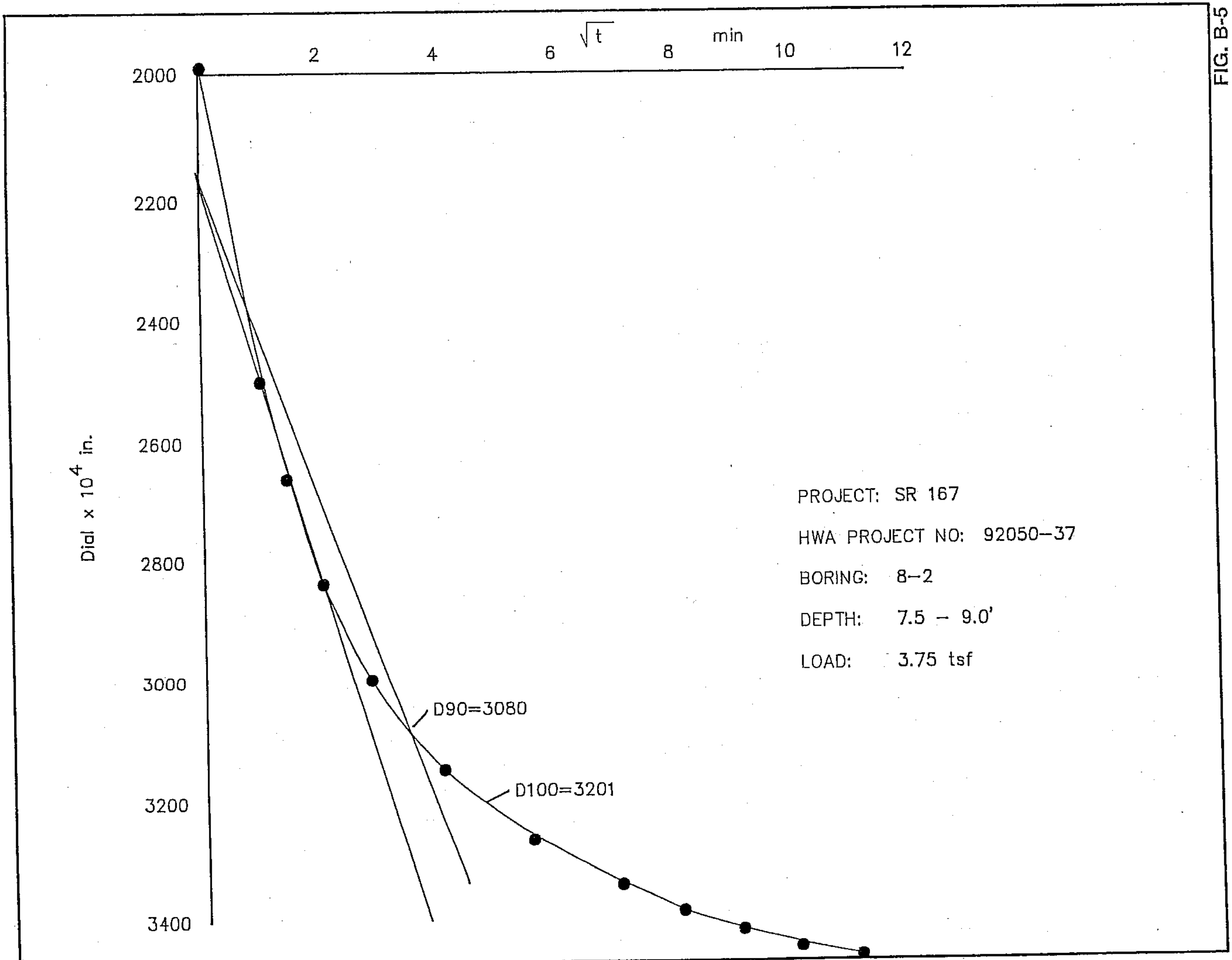
BORING: 8-2

DEPTH: 7.5 - 9.0'

LOAD: 1.75 tsf

FIG. B-4

PTY-A-001.0



$\sqrt{t}$ min
2
4
6
8
10
12
Dial x 10^4 in.
2000
2200
2400
2600
2800
3000
3200
3400
D90=3080
D100=3201
PROJECT: SR 167
HWA PROJECT NO: 92050-37
BORING: 8-2
DEPTH: 7.5 - 9.0'
LOAD: 3.75 tsf

FIG. B-5

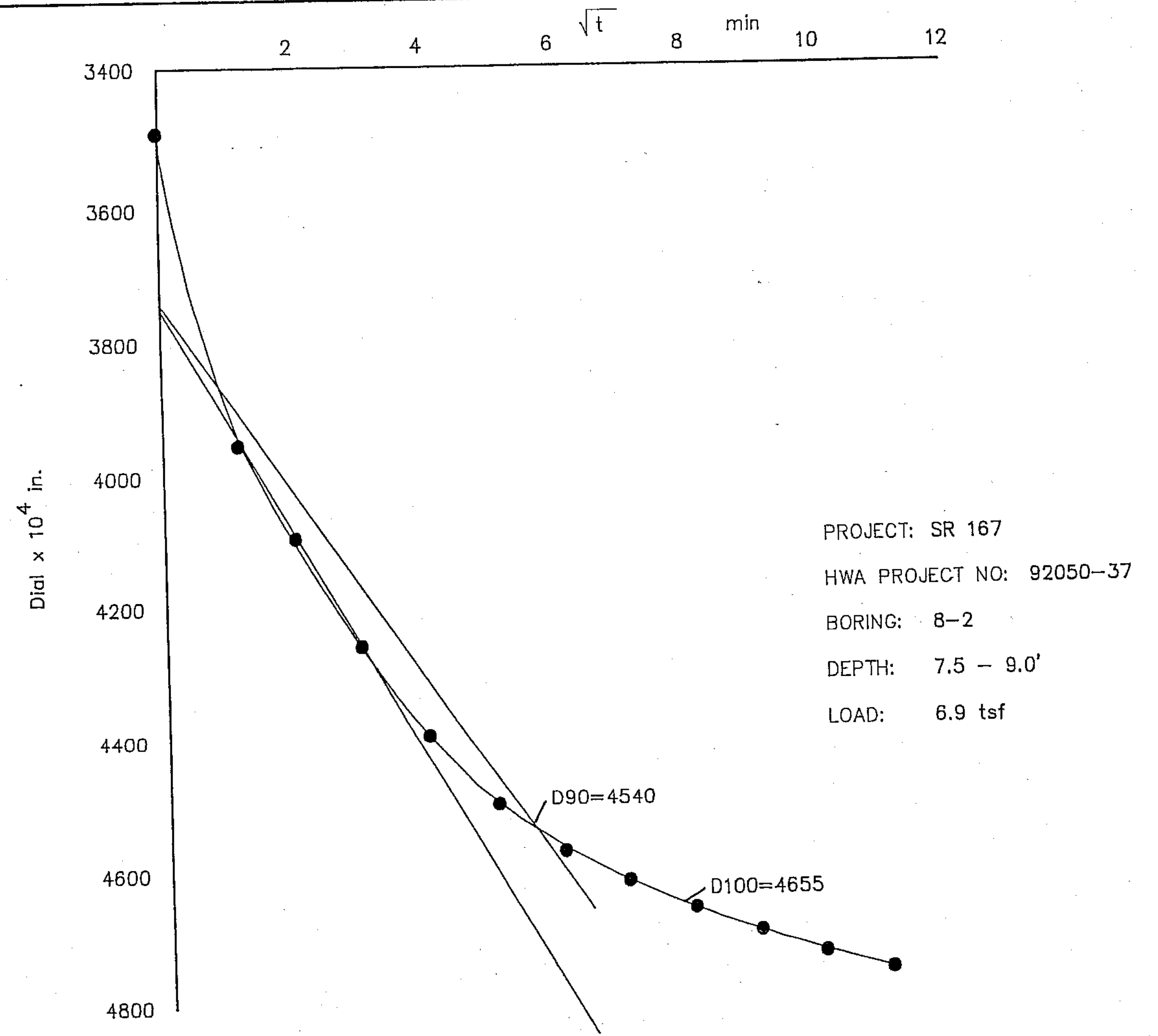

√t min
2
4
6
8
10
12
3400
3600
3800
4000
4200
4400
4600
4800
Dial x 10^4 in.
D90=4540
D100=4655
PROJECT: SR 167
HWA PROJECT NO: 92050–37
BORING: 8–2
DEPTH: 7.5 – 9.0'
LOAD: 6.9 tsf

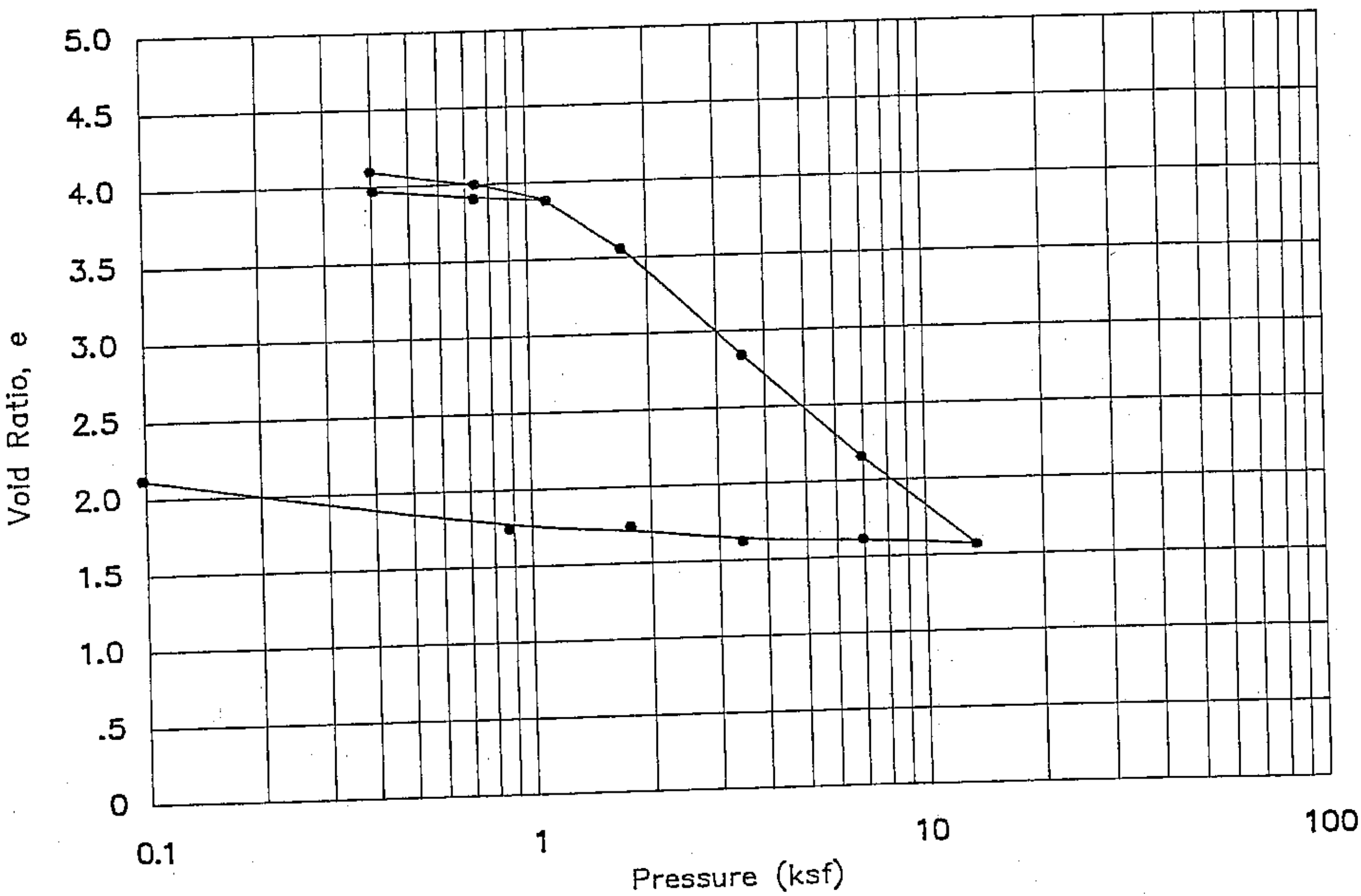
FIG. B-6

HONG WEST & ASSOCIATES, INC.

CONSOLIDATION TEST RESULTS

Project: SR 167
Location: King County, Washington
Client: Shannon & Wilson
Project Number: 92050-37
Date Tested: 12-22-92

Test No.
Boring: 8-3
Depth (ft.): 10 - 12
Diameter (in.): 2.416
Assumed Sp. Gravity: 1.9



	INITIAL	FINAL
Height (in.)	1.0	0.5935
Water Content %	207	132
Wet Density (pcf)	69.4	89.1
Dry Density (pcf)	22.6	38.4
Saturation %	94.7	104%

Sample Description: Soft, dark brown PEAT (PT)

Liquid Limit: N/A
Plastic Limit: N/A

FIG. B-7

HONG WEST & ASSOCIATES, INC.

Job No. 92050-37

Project SR-167

Date 12-21-92

Calculations for 8-2, S-4

Made By

SAMPLE DESCRIPTION

B 8-2

Sample S-4

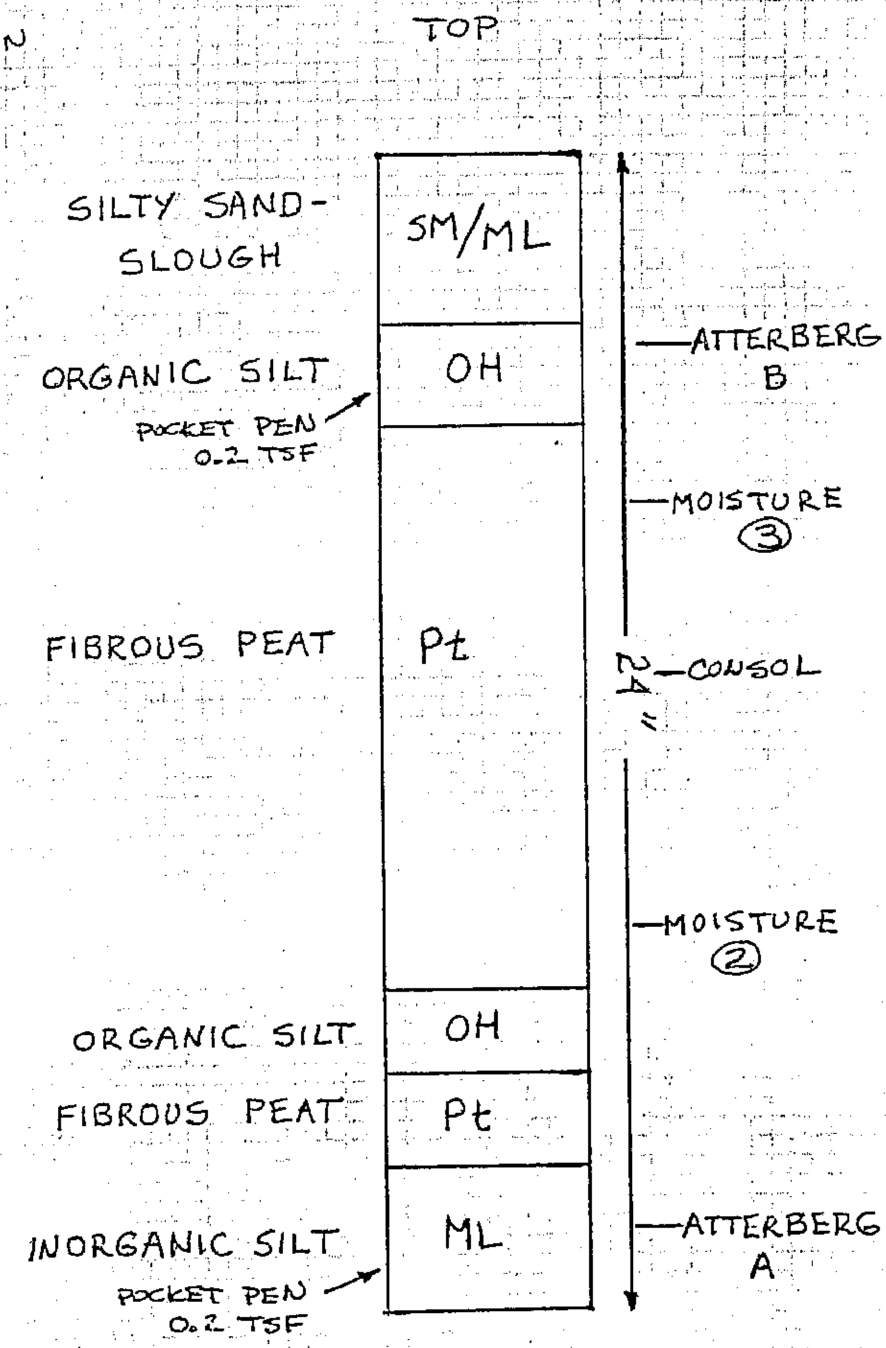


FIG. B-8

HWA HONG WEST & ASSOCIATES, INC.

Job No. 92050-37

Date 12-22-92

Project SR-167

Calculations for 8-3, S-2

Made By

SAMPLE DESCRIPTION
B 8-3
Sample S-2

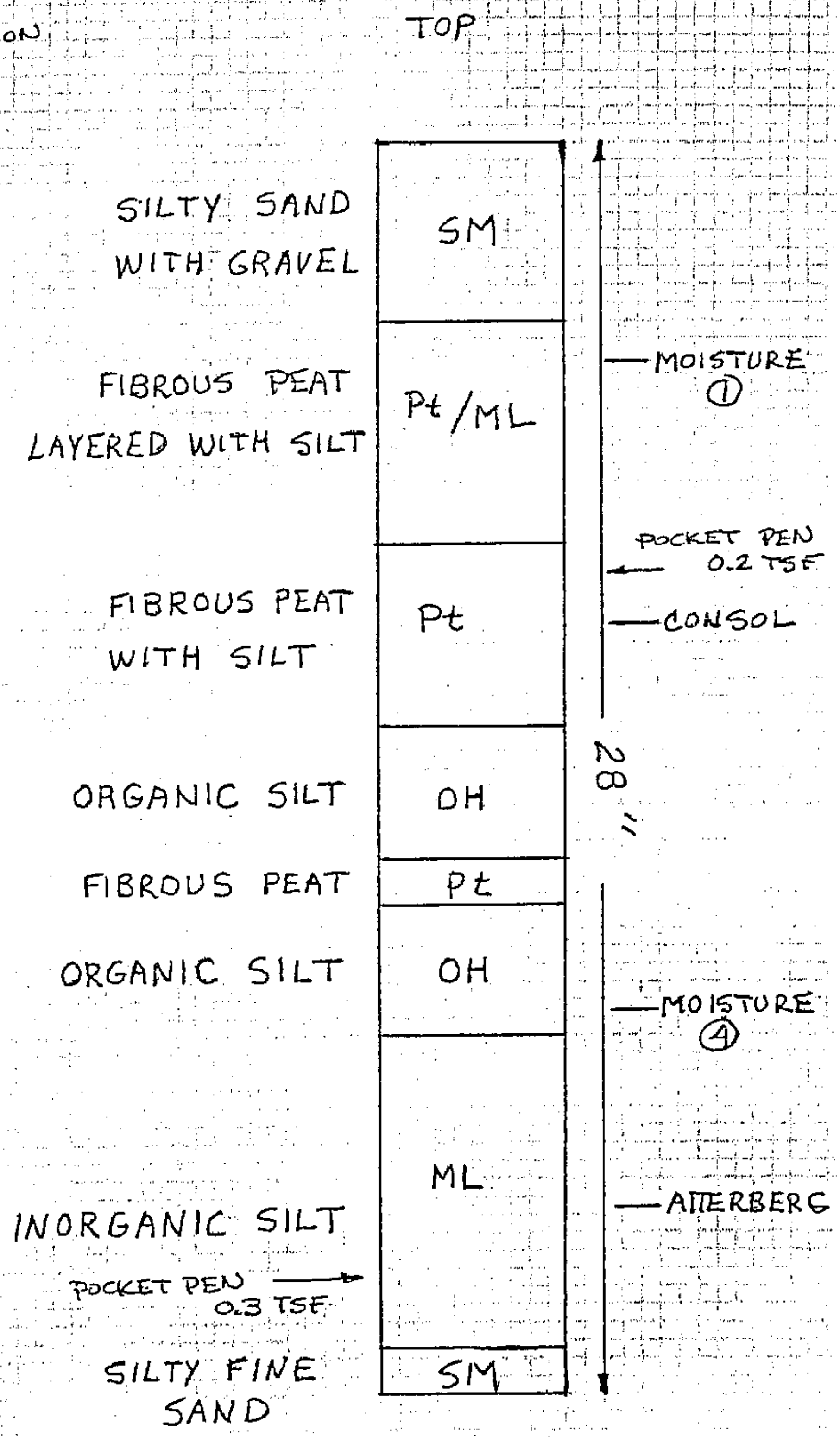


FIG. B-9

APPENDIX C

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Dated: February 22, 1993
To: Washington State Dept. of Trans.
Attn: Mr. Todd Harrison

Important Information About Your Geotechnical Engineering/ Subsurface Waste Management (Remediation) Report

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS.

Consulting geotechnical engineers prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the geotechnical engineer/geoscientist.

AN ENGINEERING REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical engineering/subsurface waste management (remediation) report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, have the consulting engineer(s)/scientist(s) evaluate how any factors which change subsequent to the date of the report, may affect the recommendations. Unless your consulting geotechnical/ civil engineer and/or scientist indicates otherwise, your report should not be used: 1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); 2) when the size, elevation, or configuration of the proposed project is altered; 3) when the location or orientation of the proposed project is modified; 4) when there is a change of ownership; or 5) for application to an adjacent site. Geotechnical/civil engineers and/or scientists cannot accept responsibility for problems which may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural changes or human influence. Because a geotechnical/waste management engineering report is based on conditions which existed at the time of subsurface exploration, construction decisions should not be based on an engineering report whose adequacy may have been affected by time. Ask the geotechnical/waste management consultant to advise if additional tests are desirable before construction starts. For example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/waste management report. The geotechnical/civil engineer and/or scientist should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST GEOTECHNICAL RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help minimize their impact. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your geotechnical engineer's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Because actual subsurface conditions can be discerned only during earthwork, you should retain your geotechnical engineer to observe actual conditions and to finalize conclusions. Only the geotechnical engineer who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The geotechnical engineer who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE GEOTECHNICAL ENGINEERING/SUBSURFACE WASTE MANAGEMENT (REMEDIATION) REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical engineering/subsurface management (remediation) report. To help avoid these problems, the geotechnical/civil engineer and/or scientist should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological and waste management findings and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE ENGINEERING/WASTE MANAGEMENT REPORT.

Final boring logs developed by the geotechnical/civil engineer and/or scientist are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical engineering/waste management reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To minimize the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/waste management report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes which aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical engineering/subsurface waste management (remediation) is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against geotechnical/waste management consultants. To help prevent this problem, geotechnical/civil engineers and/or scientists have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the engineer's or scientist's liabilities to other parties; rather, they are definitive clauses which identify where the engineer's or scientist's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your engineer/scientist will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland